GROUNDWATER LOWERING in CONSTRUCTION

A Practical Guide to Dewatering

Second Edition

Pat M. Cashman and Martin Preene

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For Lucy and Daniel

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In memoriam

Patrick Michael Cashman, the leading British exponent of groundwater control of his generation, died on June 25, 1996. For more than 40 years, during the growth of soil mechanics into the practice of geotechnical engineering, Pat was responsible, through the organizations he ran and later as a consultant, for maintaining a practical and straightforward approach to the art of groundwater control. This book, the manuscript of which was well advanced at the time of his death, sets out that approach.

Following war service with the Royal Engineers, Pat Cashman graduated from the University of Birmingham and joined Soil Mechanics Limited, soon transferring to the Groundwater Lowering Department, thus beginning his lifelong interest in this field. He became head of the department in 1961 and later became responsible for the joint venture with Soletanche, which introduced French techniques into the United Kingdom. In 1969 he became contracts director for Soletanche (UK) Limited.

In 1972 Pat joined Groundwater Services Limited (later Sykes Construction Services Limited) as managing director. Over a 10-year period, he designed and managed a huge number and range of groundwater lowering projects. Commercial and financial successes were achieved alongside technical innovation and practical advancements. In the 1980s he joined Stang Wimpey Dewatering Limited as managing director. Again, he achieved commercial success as well as introducing American ideas into British practice. During this period, Pat made a major contribution to the production of CIRIA Report 113 (*Control of Groundwater for Temporary Works*) by the Construction Industry Research and Information Association—the first comprehensive dewatering guide produced in the United Kingdom in the modern era.

In 1986, he "retired" and commenced an active role as a consultant, often working closely with Ground Water Control Limited, a contracting company formed by men who had worked for Pat during the Sykes years. His practical approach to problems meant that he was always in demand, particularly by contractors when they were in trouble. Although a practical man with a healthy suspicion of arcane theory and, in particular, computer modeling of problems, he took to the computer to document his own experience. This book is a record of a singular approach to a challenging business.

Martin Preene
Wakefield

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Preface

In the decade since the first edition of this book was published, we have perceived that groundwater lowering and dewatering activities are, thankfully, being increasingly integrated into the wider ground engineering schemes on major projects rather than being a standalone activity. There has also been a very significant increase in the number of projects where potential environmental impacts are of concern and require assessment during the development of the project.

This second edition reflects those changes. Although, as the title suggests, the primary focus of the book is still on “groundwater lowering” or “dewatering,” new material is included to provide information on more wide-ranging “groundwater control” solutions that may not rely solely on dewatering by pumping.

Specifically, new chapters have been added on cutoff methods used for groundwater exclusion (Chapter 12) and on the issues associated with permanent or long-term groundwater control systems (Chapter 14). Another notable addition is the new section in Chapter 11, which looks at groundwater control technologies used on contaminated sites—an increasingly common application.

Probably, the biggest change is the need to understand, predict, and mitigate the potential environmental impacts that can be caused by groundwater control works. This is covered in the new Chapter 15.

The remainder of the text has been updated to reflect relevant changes in technology and practice.

Acknowledgments

Preparing this edition has been a rewarding, if at times challenging, experience. I thank the editorial team at Taylor & Francis for their encouragement to complete this work.

I am grateful for the constructive and intelligent comments on parts of the text that were provided by Bob Essler, Dave Hall, Bridget Plimmer, Dan Taylor, and David Whitaker. I also thank my good friend Toby Roberts for kindly updating the afterword and providing support and comment during the preparation of this edition.

Illustrations and photographs provided by others or from published works are acknowledged in the captions, and I thank all those who provided such material and permissions.

Many new figures and illustrations have been included in this edition, and James Welch and Tomasz Jasinski have done a great job in preparing many of the new figures. I am also grateful for the help of Dyfed Evans with some of the photographic images and Lesley Power for her help with the preparation of the manuscript.

Finally, as always, I thank my family, especially my wife, Pam, for her continuous support and amazing tolerance during the gestation and birth of this book.

Martin Preene
Wakefield

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Acknowledgments to the first edition

It is difficult to make fulsome and comprehensive acknowledgments to the many who have encouraged me and thereby helped me to persist with the task of writing this book. My wife has been a steadfast "encourager" throughout its very lengthy gestation period. It was Daniel Smith who first persuaded me to compile this work. I have lost track of his present whereabouts, but I hope that he will approve of the finished work. I am grateful for his initial stimulation.

I owe many thanks to my near neighbor, Andy Belton. Without his patient sorting out of my computer usage problems, this text would never have been fit to send to a publisher. Andy will be the first to admit that this is not a subject that he has knowledge of, but having read some of the draft text and many of my faxes, on occasions, he asked some very pertinent questions. These made me think back to first principles and so have—I am sure—resulted in improved content of this work.

I thank the many organizations that have so kindly allowed use of their material, photographs, and diagrams. Where appropriate, due acknowledgment is made in the text beneath each photo or diagram. I hope that this will be acceptable to all who have helped me and have given their permissions for their material to be reproduced; a complete list of each and everyone would be formidable indeed—I do ask to be forgiven for not so doing. Following this philosophy, it would be invidious to single out and name a few of the major assistors for fear of upsetting others.

Patrick Michael Cashman
Henfield, West Sussex

At the time of Pat Cashman's death in 1996, the manuscript for this book was well advanced. I am grateful to William Powrie and the editorial team at E&FN Spon for encouraging me to complete his work. I also acknowledge the support and contribution of David Hartwell, who contributed closely with Pat on an earlier draft of the text and has provided information and assistance to me during the preparation of the manuscript.

Valuable comments on parts of the text were provided by Lesley Benton, Rick Brassington, Steve Macklin, Duncan Nicholson, Jim Usherwood, and Gordon Williams. I am also grateful to Toby Roberts for kindly writing the afterword.

Illustrations and photographs provided by others or from published works are acknowledged in the captions, and I thank all those who provided such material and permissions. The libraries of the Institution of Civil Engineers and Ove Arup & Partners have also been of great assistance in gathering together many of the references quoted herein.

Finally, I thank my wife, Pam, for her invaluable and unstinting support throughout this project.

Martin Preene
Wakefield

Chapter 1

Groundwater lowering: A personal view

Pat Cashman

Many engineering projects, especially major ones, entail excavations into water-bearing soils. For all such excavations, appropriate system(s) for the management and control of the groundwater and surface water runoff should be planned before the start of each project. In practice, this can only be done with knowledge of the ground and groundwater conditions likely to be encountered by reference to site investigation data. The control of groundwater (and also surface water runoff) is invariably categorized as "temporary works" and, therefore, is almost always regarded by the client and the engineer or architect as the sole responsibility of the contractor and of little or no concern of theirs. In many instances, this philosophy has been demonstrated to be shortsighted and ultimately costly to the client.

Sometimes, as work proceeds, the actual soil and groundwater conditions encountered may differ from what was expected. Should this happen, all concerned should be willing to consider whether to modify operations and construction methods as the work progresses and as more detailed information is revealed. Based on this philosophy, I advocate, particularly for large projects, frequent "engineering-oriented" reappraisal meetings between client/owner and contractor or both (as distinct from "cost-oriented" meetings). This will afford the best assurance that the project will be completed safely, economically, and within a realistic program time and cost.

On a few occasions, I have been privileged to be involved in the resolution of some difficult excavation and construction projects in which the engineer succeeded in persuading the client to share the below ground risks with the contractor. During the progress of the contract, there were frequent engineering-oriented meetings with the contractor to discuss and mutually agree on how to proceed. I believe that the engineers concerned with these complex projects realized that it would not be in the best interests of their client to adhere rigidly to the traditional view that the contractor must take all of the risk. They were enlightened, had a wealth of practical experience, and, therefore, had a realistic awareness that the soil and groundwater conditions likely to be encountered were complex. In addition, they realized that the measures for attaining stable soil conditions during construction may not be straightforward. The few occasions when I have experienced this joint risk-sharing approach have, without exception in my view, resulted in sound engineering solutions to problems that needed to be addressed: they were resolved sensibly and the projects were completed within realistic cost to the client. Furthermore, claims for additional payments for dealing with unforeseen conditions were not pressed by the contractor.

I found these experiences most interesting and enlightening, and I learned much by having direct access to different points of view of the overall project as distinct from our own view as a specialist contractor. I find it encouraging that, in recent years, the target form of contract—the client and the contractor sharing the risks of unforeseen conditions—is being implemented more frequently. Thereby, the contractor is confident of a modest but reasonable profit, and the client is not eventually confronted with a multitude of claims for additional payments, some of which may be spurious, but all requiring costly time-consuming analysis and investigation.

There are three groups of methods available for temporary works control of groundwater:

1. Lowering of groundwater levels in the area of construction by water abstraction; in other words, groundwater lowering or dewatering.
2. Exclusion of groundwater inflow to the area of construction by some form of very low permeability cutoff wall or barrier (e.g., sheet piling, diaphragm walls, or artificial ground freezing).
3. Application of a fluid pressure in confined chambers such as tunnels, shafts, and caissons to counterbalance groundwater pressures (e.g., compressed air or earth pressure balance tunnel-boring machines).

This book deals primarily with group 1 methods, i.e., the groundwater lowering and dewatering methods. The various methods of all three groups, their uses, advantages, and disadvantages are presented in summary form in Chapter 5. Chapter 12 specifically discusses group 2 methods, i.e., groundwater exclusion methods.

Rudolf Glossop (1950) stated

> "The term drainage embraces all methods whereby water is removed from soil. It has two functions in engineering practice: permanent drainage is used to stabilize slopes and shallow excavations; whilst temporary drainage is necessary in excavating in water bearing ground."

This book principally addresses the subject of temporary drainage, although many of the principles are common to both temporary and permanent requirements.

The book is intended for use by the practical engineer (contractor, consultant, or client); however, it is intended particularly for the guidance of the specialist "dewatering practitioner" or advisor. In addition, it is recommended to the final year graduate or master's student reading civil engineering or engineering geology, as well as to the civil engineering–oriented hydrogeologist. It is deliberately addressed to the practitioner involved in the many day-to-day small- to medium-scale dewatering projects for which a simplistic empirical approach is usually adequate. It is anticipated that the typical reader of this work will be one quite comfortable with this philosophy but one who is aware of the existence of—although perhaps wishing to avoid—the purist hydrogeologist philosophy and the seemingly unavoidable high-level mathematics that come with it.

We, the writers and the readers, are pragmatic temporary works engineers—or, in the case of some readers, aspiring to be so—seeking the successful and economic completion of construction projects. For the small- and medium-sized projects (which are our "bread and butter"), there seems little practical justification for the use of sophisticated and time-consuming techniques when simpler methods can give serviceable results. The analytical methods described in this book are based on a large amount of field experience by many practitioners from several countries and have thereby been proven to be practicable and adequate for most temporary works assessment requirements. I consider that J. P. Powers et al. (2007) stated a great dewatering truism: "The successful practitioner in dewatering will be the person who understands the theory and respects it but who refuses to let theory overrule judgment."

The Dupuit–Forcheimer analytical approaches are used extensively here. I am conscious that purists will question this simplistic approach. Our riposte, based on approximately 30 or more years of dealing with groundwater lowering problems, is that, in our experience and that of many others, this empirical philosophy has resulted in acceptably adequate pumping installations, always provided, of course, that due allowance is made for the often limited reliability of ground and groundwater information available. It requires acquired practical field experience to assess the quality of the site investigation data. Whenever possible, reference should be made to other excavations in adjacent areas or in similar soil conditions to verify one's proposals. In the text, there are some brief descriptions of a number of relevant case histories that the authors have dealt with in the past.

I readily acknowledge that, for a groundwater lowering system design pertinent to large-scale and long-term projects (for example, the construction of a dry dock, a nuclear power station, or an open pit mining project), more sophisticated methods of analysis will be appropriate. These can provide reassurance that the pragmatic solution is approximately correct, but do we ever know the "permeability value" to a similar degree of accuracy?

The underlying philosophy of this publication is to address the pragmatic approach. It follows that the following three questions arise:

- How does water get into the ground, and how does it behave while getting there and subsequently behave while there?
- What is the interrelationship between the soil particles and the groundwater in the voids between them?
- How do we control groundwater and surface water runoff and prevent it from causing problems during excavation and construction?

A thorough site investigation should go a long way to providing the answers to these questions. Unfortunately, experience indicates that many engineers responsible for specifying the requirements for project site investigations consider only the *designer's* requirements and do not address the other important considerations, i.e., *how can this be built?* Oftentimes, the site investigation is not tailored to obtaining data pertinent to temporary works design requirements or to problems that may occur during construction.

The contractor should not always expect to encounter conditions exactly as revealed by the site investigation. Soils, because of the very nature of their deposition and formation, are variable and rarely, if ever, isotropic and homogeneous, as is assumed in many of the analytical methods. The contractor should carry out the works using their professional skills and abilities and should be prepared to adjust, if changes in circumstances are revealed, as the work proceeds.

Throughout the planning, excavation, and construction phases of each project, safety considerations must be of paramount importance. Regrettably, the construction industry historically has a poor safety performance record.

Let us consider another professional discipline. Hopefully, no surgeon would contemplate commencing an operation on a patient without carrying out a thorough physical examination and having the information from X-ray, electrocardiogram, urine, blood, and other test results and any pertinent scans available beforehand. The surgeon will realize that these may not indicate everything and that, during the operation, complications may occur but the possibilities of such "surprise" occurrences will have been minimized by having reliable site investigation data concerning the patient.

Likewise, if the client's engineer or designer provides comprehensive ground and groundwater information at a tender stage, the surprise occurrences during construction could be minimized. An experienced contractor, with the cooperation of an experienced client or engineer, should be able to agree on how to adjust working techniques to deal adequately with the changed circumstances as or if they are revealed, and this at a realistic final cost.

1.1 STRUCTURE OF THE REST OF THE BOOK

At the commencement of each chapter, the introduction acts as a summary of the subject matter to be covered therein. I hope that this approach will enable the readers to decide speedily which chapters they wish to read forthwith when seeking guidance on how to deal with their individual requirements and which chapters may be deferred until later.

Chapter 2 contains a brief historical review of the principal theories concerning seepage toward wells and of the technologies used to apply them. Many readers will probably consider this as superfluous and omit it from their initial reading. As their interest in this subject becomes further stimulated, we hope that they will turn back to it. I derived great pleasure when researching this aspect some years ago.

Chapter 3 contains a very brief summary of the hydrological cycle (i.e., how water gets into the ground), together with a similarly brief summary concerning soils, their properties, and permeability. The theoretical concepts of Darcy's law and the nature of flow to wells are introduced, which is used in a practical sense later. Groundwater chemistry is also briefly discussed.

Chapter 4 discusses the mechanisms of instability problems in excavations. An understanding of these issues can be vital when selecting a groundwater control method.

Chapter 5 presents, in summary form, the principal features of the methods available for control of groundwater by exclusion and by pumping.

Chapter 6 addresses site investigation requirements, but only those specific to groundwater lowering, and does not detail the intricacies of the available methods of site investigation. This chapter also covers permeability and its determination in the field and laboratory. Guidance is also given on the relative reliability of permeability estimated by the various methods.

Chapter 7 describes various empirical and simple design methods for assessing the discharge flow rates required for groundwater lowering installations and for determining the number of wells. Several simple design examples are included in an appendix.

Chapters 8 through 11 address the various groundwater lowering methods: sump pumping, wellpointing, deep wells, and less commonly used methods.

Chapter 12 describes the various techniques that can be used to form groundwater cutoff walls or barriers to exclude groundwater from excavations.

Chapter 13 describes the types of pumps suitable for the various systems.

Chapter 14 describes the particular issues and challenges associated with permanent (or at least very long term) dewatering and groundwater control systems.

Chapter 15 deals with some of the environmental effects of groundwater lowering, including ground settlements. Various approaches to the assessment and mitigation of environmental effects are also discussed.

Chapter 16 presents appropriate methods of monitoring and maintenance to ensure that groundwater lowering systems operate effectively when they are first installed and after extended periods of operation.

Chapter 17 covers safety, contractual, and environmental regulation issues (using British convention but based on principles of good practice that are applicable in other countries).

Chapter 18 is a short afterword (contributed by the eminent dewatering engineer Toby Roberts), looking at the future of groundwater lowering.

The book ends with a comprehensive list of references, a glossary of terms pertinent to the subject matter, and also a summary of the symbols and notations used. Various appendices are included, providing detailed background information on various subjects.

Finally, I reiterate our earlier statement: this book is intended for the guidance of practical engineers and those—many we hope—desirous of joining us. I trust that there are several new aspirants who will realize, after reading this book, that it is a challenging scientific field and, to some extent, an applied art form as well, wherein the requirement for practical experience is paramount. No two sites have the same requirements—this is one of its fascinations!!!

Chapter 2

History of groundwater theory and practice

2.1 INTRODUCTION

Man has been aware of groundwater since prehistory, long before Biblical times. Over the centuries, the mysteries of groundwater have been solved, and man has developed an increasing capability to manipulate it to his will.

This chapter describes some of the key stages in the development of the understanding and control of groundwater. The history of some of the technology now used for groundwater lowering is also discussed, especially in relation to early applications in the United Kingdom. Detailed knowledge of the history of groundwater control might not be considered essential for a practical engineer working today. Nevertheless, a study of the past can be illuminating, not least by showing that even when theories are incomplete, and the technology untried, the application of scientific principles and engineering judgment can still allow groundwater to be controlled.

2.2 EARLIEST TIMES TO THE SIXTEENTH CENTURY

The digging of wells for the exploitation of water and primitive implementation of water management dates back to Babylonian times and even earlier.

The source of the water flowing from springs and in streams was a puzzling problem and the subject of much controversy and speculation. It was generally held that the water discharged from springs could not be derived from rain. Ingenious hypotheses were formulated to account for the occurrence of springs. Some early writers suggested large inexhaustible reservoirs, whereas others recognized that there must be some form of replenishment of the supplying reservoirs. The Greek hypotheses, with many incredible embellishments, were generally accepted until near the end of the seventeenth century. The theory of rainfall infiltration was propounded by only a very few writers.

Although the Romans and other early cultures indulged in quite-sophisticated water management projects by building imposing aqueducts to channel water from springs and other sources to centers of population, they had no understanding of the sources of replenishment of groundwater. The aqueducts of the Romans were remarkable and showed great appreciation of the value of water, but their methods of measuring or estimating water quantities were crude. Generally, they seem to have lacked any semblance of knowledge of either surface or groundwater hydrology.

According to Tolman (1937)

> "Centuries have been required to free scientists from superstitions and wild theories handed down from the dawn of history regarding the unseen subsurface water… even in this century, there is still much popular superstition concerning underground water."

An elemental principle, that gravity controls the motion of water underground as well as at the surface, was not appreciated by all. There existed a popular belief that "rivers" of underground water passed through solid rock devoid of interconnected interstices and flowed under intervening mountain ranges.

Marcus Vitruvius, a Roman architect who lived during the time of Christ, produced a book describing the methods of finding water. He wrote of rain and snow falling on mountains and then percolating through the rock strata and reappearing as springs at the foot of the mountains. He provided a list of plants and other conditions indicative of groundwater, such as color and dampness of the soil and mists rising from the ground early in the morning. Vitruvius and his two contemporaries, Cassiodorus and Plinz, were the first to make serious efforts to list practical methods for locating water, and this was when geology was yet unknown.

2.3 RENAISSANCE PERIOD TO THE NINETEENTH CENTURY

At the beginning of the sixteenth century, Leonardo da Vinci directed his attention to the occurrence and behavior of water. He correctly described the infiltration of rain and the occurrence of springs and concluded that water goes from rivers to the sea and from the sea to the rivers and, thus, is constantly circulating and returning. At about the same time, Pallissy, a French Huguenot, presented a clear and reasoned argument that all water from springs is derived from rain.

The latter part of the seventeenth century was a watershed in the beginnings of an understanding of the replenishment of groundwater. Gradually, there arose the concept of a "hydrological cycle." This presumed that water was returned from the oceans by way of underground channels below the mountains. The removal of salt was thought to be either by distillation or by percolation and there were some highly ingenious theories of how water was raised up to the springs.

2.4 PROGRESS FROM A QUALITATIVE TO A QUANTITATIVE SCIENCE

It was in the seventeenth century that the quantitative science of hydrology was founded by Palissy, Pérrault, and Mariotté in France; Ramazzini in Italy, and the astronomer Halley in Britain.

Palissy, a sixteenth-century potter and paleontologist, stated that rain and snowmelt were the only source of spring and river waters and that rainwater percolates into the earth, following "a downward course until they reach some solid and impervious rock surface over which they flow until they make some opening to the surface of the earth."

Pérrault made rainfall runoff measurements and demonstrated the fallacy of the long-held view that rainfall was not sufficient to account for the discharge from springs. He also measured and investigated evaporation and capillarity. Mariotté verified Pérrault's results and showed that the flow from springs increased in rainy weather and decreased in times of drought. Halley made observations of the rate of evaporation from the Mediterranean Ocean and showed that this was adequate to supply the quantity returned to that sea by its rivers. His measurements of evaporation were conducted with considerable care but his estimates of stream flow were very crude.

Toward the end of the eighteenth century, La Metherie extended the research of Mariotté and brought this to the attention of meteorologists. He also investigated permeability and explained that some rain flows off directly (surface water runoff), some infiltrates into the top soil layers only and evaporates or feeds plants, whereas some rain penetrates underground from where it can issue as springs (i.e., infiltration or groundwater recharge). This is the first recorded mention of "permeability" and, therefore, is the first link between hydrology and seepage to wells.

2.4.1 Seepage toward wells

The robust Newcomen engine greatly influenced mining practice during the eighteenth century, but it was far too cumbersome for construction works. Generally speaking, until the early nineteenth century, civil engineers, by the use of timber caissons and other devices, avoided pumping whenever possible. However, where there was no alternative, pumping was usually done by hand, which is a very onerous task, using a rag and chain pump (Figure 2.1), also known as "le chaplet" (the rosary).

Figure 2.1 Manually operated rag and chain pump (1556). The balls, which are stuffed with horsehair, are spaced at intervals along the chain and act as one-way pistons when the wheel revolves. (From Bromehead, C.N., *A History of Technology*. Singer, C., Holmyard, E.J., Hall, A.R., and Williams, T.L., eds. Oxford University Press, Oxford, 1–40, 1956. By permission of Oxford University Press.)

Some idea of the magnitude of the problem is given by de Cessart in his book *Oeuvres Hydrauliques*. Speaking of the foundations for the abutment of a bridge at Saumur in 1757, he says that 45 chain pumps were in use and operated by 350 soldiers and 145 peasants. Work on this type of pump was, of course, most exhausting, and the men could only work in short spells. Pryce, in his *Mineralogea Cornubiensis* in 1778, said that work on pumps of this sort led to a great many premature deaths among Cornish miners.

For permanent installations such as graving docks, large horse-driven chaplet pumps were used. Perronnet, the famous French bridge builder, used elaborate pumping installations in the cofferdams for the piers of his larger bridges; for example, undershot water wheels were used to operate both chaplet pumps and Archimedean screws.

According to Crèsy, writing in 1847, the first engineer to use steam pumps on bridge foundations was Rennie, who used them on the Waterloo Bridge in 1811. In the same year, Telford, on the construction of a lock on the Caledonian Canal at Clachnacarry, at first used a chain pump worked by six horses but replaced it with a 9-hp steam engine pump. From then on, steam pumps were used during the construction of all the principal locks on that canal. In 1825, Marc Brunel used a 14-hp steam engine when sinking the shafts for the Thames Tunnel. By this date, steam pumping seems to have been the common practice for dealing with groundwater, and thus, below ground excavations for construction were less problematical.

2.4.2 Kilsby Tunnel: London to Birmingham Railway

There seems to have been no important advance in pumping from excavations until the construction in the 1830s by the renowned civil engineer Robert Stephenson of the Kilsby Tunnel south of Rugby, on the London to Birmingham Railway (Preene 2004). He pumped from two lines of wells sited parallel to and on either side of the line of the tunnel drive (Figure 2.2).

It is clear from Stephenson's *Second Report to the Directors of the London, Westminster and Metropolitan Water Company* (1841) that he

Figure 2.2 Pumps for draining the Kilsby Tunnel. A pumping well is shown in the foreground, with the steam pumphouse in the distance. (From Bourne, J.C., *Drawings of the London and Birmingham Railway*. Collection of the Library of the Institution of Civil Engineers, London. Courtesy of the Institution of Civil Engineers.)

was the first to observe and explain the seepage or flow of water through sand to pumped wells. The wells were sited just outside the periphery of the construction to lower the groundwater level in the area of the work by pumping from these water abstraction points. This is most certainly the first temporary works installation of a deep well groundwater lowering system in Britain, if not in the world. The following extract (Boyd-Dawkins 1898; courtesy of the Institution of Civil Engineers Library) quotes from the report and shows that Stephenson had understood the mechanism of groundwater flow toward a pumping installation:

> "The Kilsby Tunnel, near Rugby, completed in the year 1838, presented extreme difficulties because it had to be carried through the water-logged sands of the Inferior Oolites, so highly charged with water as to be a veritable quicksand. The difficulty was overcome in the following manner. Shafts were sunk and steam-driven pumps erected in the line of the tunnel. As the pumping progressed, the most careful measurements were taken of the level at which the water stood in the various shafts and boreholes; and I was soon much surprised to find how slightly the depression of the water level in the one shaft, influenced that of the other, not withstanding a free communication existed between them through the medium of the sand, which was very coarse and open. It then occurred to me that the resistance which the water encountered in its passage through the sands to the pumps would be accurately measured by the angle or inclination which the surface of the water assumed toward the pumps, and that it would be unnecessary to draw the whole of the water off from the quicksands, but to persevere in pumping only in the precise level of the tunnel, allowing the surface of the water flowing through the sand to assume that inclination which was due to its resistance.
>
> The simple result of all the pumping was to establish and maintain a channel of comparatively dry sand in the immediate line of the intended tunnel, leaving the water heaped up on each side by the resistance which the sand offered to its descent to that line on which the pumps and shafts were situated."

As Boyd-Dawkins then comments

> "The result of observations, carried on for 2 years, led to the conclusion that no extent of pumping would completely drain the sands. Borings, put down within 200 yards (185 m) of the line of the tunnel on either side, showed further, that the water level had scarcely been reduced after 12 months continuous pumping and, for the latter 6 months, pumping was at the rate of 1,800 gallons per minute (490 m^3/h). In other words, the cone of depression did not extend much beyond 200 yards (185 m) away from the line of pumps.

> In this account, ... it is difficult to decide which is the more admirable, the scientific method by which Stephenson arrived at the conclusion that the cone of depression was small in range, or the practical application of the results in making a dry (the authors would have used the word "workable" rather than "dry") pathway for the railway between the waters heaped up (in the soil)... on either side."

It is astonishing that neither Robert Stephenson nor any of his contemporaries realized the significance of this newly discovered principle. That is, by sinking water abstraction points and, more importantly, by placing them clear of the excavation so that the flow of water in the ground will be away from the excavation rather than toward it, stable ground conditions were created. For many decades, this most important principle was ignored.

2.4.3 Early theory—Darcy and Dupuit

In the 1850s and early 1860s, Henri Darcy made an extensive study of the problems of obtaining an adequate supply of potable water for the town of Dijon. He is famous for his Darcy's law (Darcy 1856), postulating how to determine the permeability of a column of sand of selected grading, knowing the rate of water flow through it, but this formed only a small part of his treatise. He compiled a very comprehensive report (two thick volumes) in which he analyzed the available sources of water from both rivers and wells—some of them artesian—and how to economically harness all these for optimum usage.

Darcy investigated the then current volume of supply of water per day per inhabitant for about 10 municipalities in Britain—Glasgow, Nottingham, and Chelsea, among others—as well as Marseille and Paris, and many other French towns. He concluded that the average water provision in Britain was 80–85 L/inhabitant per day and more than 60 L/inhabitant per day for Paris. Darcy designed the water supply system for Dijon on the basis of 150 L/inhabitant per day—no doubt his Victorian contemporaries this side of "la manche" would have applauded this philosophy.

In the mid-1860s, Dupuit (1863), using Darcy's law to express soil permeability, propounded his equations for determining flow to a single well positioned in the middle of an island. Dupuit made certain simplifying assumptions and, having stated them (i.e., truly horizontal flow), then discounted their implications. For this, Dupuit has been much castigated by some later purists, but most accept that the Dupuit concept, later slightly modified by Forcheimer, is acceptable and adequate in many practical situations.

The exchange of information was not as simple in Darcy and Dupuit's time as it is now. Much of the fundamental work of these two French engineers was duplicated by independent developments shortly afterward in Germany, Austria, and, a little later, in the United States.

By about 1883, Reynolds demonstrated that, for linear flow, that is, flow in orderly layers, commonly known as laminar flow, there is a proportionality existing between the hydraulic gradient and the velocity of flow. This is in keeping with Darcy's law, but as velocity increased, the pattern of flow becomes irregular (i.e., turbulent), and the hydraulic gradient approached the square of the velocity. Reynolds endorsed the conclusion that Darcy's law gives an acceptable representation of the flow within porous media, that is, the flow through the pore spaces of soils will remain laminar, save for very rare and exceptional circumstances. However, this may not always be true of flow through jointed rocks (e.g., karstic limestone formations).

2.5 LATER THEORETICAL DEVELOPMENTS

In his Rankine Lecture, Glossop (1968) suggested that "classical soil mechanics" was founded on the work of Terzaghi, which dates from his first book published in 1925, and that it was strongly influenced by geological thinking. In 1913, Terzaghi published an article dealing with the hydrology of the Karst region after studying the geology for a hydroelectric scheme in Croatia. He soon realized that geology would be of far more use to engineers if the physical constants of rocks and soils were available for design (Terzaghi 1960). This was the first positive marrying of civil engineering and geology.

2.5.1 Verifications and modification of Darcy

In 1870, a civil engineer in Dresden, Adolph Theim, reviewed Darcy's experiments and went on to derive the same equations as Dupuit's for gravity and artesian wells. Theim was the first to collect extensive field data in support of his and Dupuit's equations. Thus, he was the first researcher to apply practical field experience to test the validity of the analytical pronouncements. This fundamentally practical philosophy was to be typical of Terzaghi and other later pioneers of soil mechanics, in which reliable field measurements were essential to verify assumptions.

The next contributor to the advance of groundwater flow theory was an Austrian engineer, Forcheimer. In the late nineteenth century, he published his first article on flow toward wells (Forcheimer 1886). Over a period of half a century, he published many contributions to this field of technology. Based on an analogy with heat flow, he developed the use of flow nets in tracing the flow of water through sand. His results were published in 1917 and certainly influenced Terzaghi. In addition, Forcheimer undertook the analysis of gravity flow toward a group of wells and introduced the concept of a hypothetical equivalent single well. This concept, later endorsed by Weber, is of great practical importance to this day in analyzing well systems associated with civil engineering projects for the construction of temporary works.

In the 1950s, an Australian research worker, Chapman (1959), investigated the problem of analyzing long lines of closely spaced wells or wellpoints. Building on earlier theoretical work by Muskat (1935), who studied plane seepage through dams, Chapman's modeling and analytical work produced solutions for single and double lines of wells or wellpoints. These solutions are still widely used today for the analysis of wellpoint systems for trench excavations.

2.5.2 Nonsteady-state flow

An important aspect of the theory of flow toward wells, on which the various investigators in Europe were relatively silent, was nonsteady-state flow. The first European investigator to produce anything of importance was Weber (1928). His work is still one of the most complete treatments of the subject. Weber, like Forcheimer before him, was very thorough in gathering field performance data and correlating it with his theoretical analyses.

In 1942, Meinzer, Chief of the Ground Water Division of the United States Geological Survey, edited a comprehensive outline on the development of fieldwork and theoretical analysis in groundwater hydrology up to that time (Meinzer 1942). His contributions had already extended over more than a decade. It was Meinzer who encouraged engineers, physicists, and mathematicians, as well as geologists, to undertake the challenging responsibilities delegated to his division. Meinzer is regarded by many as the first modern hydrogeologist and, in this specialist field, is considered as its "father" in like manner that Terzaghi is considered the father of soil mechanics. Theis is probably the best known of many of Meinzer's protégés. Theis approached the treatment of nonsteady-state flow from a different angle to that of Weber. His conceptual approach (Theis 1935) is now almost universally used as the basis for nonsteady-state flow analysis.

During the early 1930s, Muskat, Chief Physicist for the Gulf Research and Development Company, was the leader of a team that compiled a comprehensive and scholarly volume treating all phases of flow of fluids through homogeneous porous media (Muskat 1937). The work was concerned primarily with the problems involved in the flow of oil and oil–gas mixtures through rocks and sand. Muskat's approach was consistently that of a theoretical physicist rather than that of a field engineer. His appraisals of analytical methods of test procedures tended to be more that of the scientific purist than of one concerned with the practical pragmatic needs in the field. However, the accomplishments of Muskat and his colleagues have been of immeasurable value to an understanding of seepage flow.

Muskat made much use of electric analogue models and sand tank models. He investigated the effects of stratification and anisotropy and developed the transformed section. He made extensive studies of multiple confined wells. He seems to have given much serious thought to the limited validity of the Dupuit equations and pronounced, "Their accuracy, when valid, is a lucky accident." No one before or since has been so intolerant of Dupuit. Boulton (1951) and many other investigators do not subscribe to Muskat's dismissiveness of the practical usefulness of the Dupuit–Forcheimer approach.

2.6 GROUNDWATER MODELING

In a pedantic context, groundwater modeling involves the use of models or analogues to investigate or simulate the nature of groundwater flow. In modern parlance, groundwater modeling invariably refers to numerical models run on computers. In fact, these are not true models but iterative mathematical solutions to a model or mesh that is defined by the operator; a solution is considered to be acceptable when the errors reach a user-defined level. These models have been developed by many organizations from original esoteric research tools into the current generation of easier to use models with excellent presentation of results that, when used appropriately, can clearly demonstrate what is happening to the groundwater flow. The use of numerical modeling is described in Section 7.10.

The origins of groundwater models and analogues are to be found not in groundwater theory but in other scientific fields such as electricity and heat flow. In the mid-nineteenth century, Kirchoff studied the flow of electrical current in a thin disk of copper. However, it is not clear who recognized that the mathematical expressions or laws governing the flow of electricity were analogous to those governing the flow of thermal energy and groundwater.

This dawning of the electrical analogy to groundwater flow was given a major impetus in North America, where growing interest in the theoretical aspects of oil reservoir development led to major developments. As noted earlier, the significant contributions of Muskat during the 1930s included electrical analogues and sand tank models. In 1935, Wyckoff, who had worked with Muskat, published the first conductive paper model study of groundwater flow through a dam. The use of conductive paper was a direct development from Kirchoff's original work, nearly a century earlier, and became the practical basis for much of the two-dimensional isotropic analogue modeling for the next fifty years before numerical models finally superseded it. The conductive paper technique was given a further boost with the development of Teledeltos paper in 1948. This aluminum-coated, carbon-based paper enabled rapid two-dimensional studies to be undertaken.

Karplus (1958) presents an extensive discussion on the use and limitations of conductive paper models, as well as introducing resistance networks in which the uniform conducting layer is replaced by a grid of resistors. The advantage of this system was that it could be developed into three dimensions, and therefore, the problems of making measurements within a solid were overcome. Herbert and Rushton (1966) developed resistance networks to introduce switching techniques to determine free water surface, transistors to simulate storage, and also evolved time-variant solutions. Case histories of the practical application of resistance networks have been published for pressure relief wells under Mangla Dam in Pakistan (Starr et al. 1969) and for a deep well system at Sizewell B Power Station in England (Knight et al. 1996).

Other analogues include electrolytic tanks in which a thin layer of conducting fluid is used instead of the conducting paper or a stretched rubber membrane that, when distorted at right angles to the surface, forms a shape analogous to the cone of depression formed by a well. Neither of these techniques or others seems to have been used extensively for the solution of practical groundwater problems.

A different class of model is the sand tank type (Cedergren 1989). These physical models consist of two parallel plates of clear material such as glass, closely spaced and filled with sand to represent the aquifer. Physical impediments to flow, such as dams or wells, can be inserted, and the model can be saturated. By injecting a tracer or dye on the upstream side, the flow path(s) can be observed. The viscous flow or Hele-Shaw technique—so named after the man who first used this technique in England—is a variation to the sand tank. These techniques have been used regularly to demonstrate the form of groundwater flow, particularly beneath dams, but have limited application to groundwater lowering in construction.

2.7 EARLY DEWATERING TECHNOLOGY IN BRITAIN

Much of the technology that forms the basis of modern dewatering methods was developed in the United States or Germany and was introduced to Britain in the early part of the twentieth century. Up to that time, any groundwater lowering required was achieved by the crude (but often effective) methods of pumping from timbered shafts or from open-jointed subdrains laid ahead and below trench or tunnel works.

During the nineteenth century, early generations of groundwater exclusion methods had been developed in the mining industry in the form of close-fitting "tubbing" used to line shafts and keep water out. These tubbings were initially made from timber and, later, from cast iron. In the very early years of the twentieth century, British mines made the first use of artificial ground freezing (in which chilled brines were used to freeze groundwater), and the "cementation" method in which cement-based grouts were used to prevent groundwater flow through pores and fissures in soils and rocks (Younger 2004). These techniques were soon added to the repertoire of civil engineers.

The wellpoint method is probably the oldest of the modern dewatering pumping techniques. Originally, wellpoints were a simple form of driven tube, developed by Sir Robert Napier on his march to Magdala in 1896, during his Abyssinian campaign. Each Abyssinian tube, as they were known, was driven to depth using a sledgehammer. If water was found, the tube was then equipped with a conventional village type hand pump. According to Powers et al. (2007), wellpoints were used for dewatering in North America from 1901, but the modern form of the method probably derives from equipment developed by Thomas Moore in New Jersey in 1925. His equipment was an advance in that installation was by jetting to form a clean hole that was backfilled with filter sand. His system, known as the Moretrench equipment, is identical in principle to that used today; indeed, the Moretrench Corporation is still in the dewatering business more than 85 years after Thomas Moore's original innovation.

In Britain, civil engineer H. J. B. (later Sir Harold) Harding was one of the leading practitioners in the new art of geotechnology, which included groundwater lowering. In the 1930s, working for John Mowlem & Company, Harding was the contractor's agent on the Bow–Leyton extension of the London Underground central line. Here, he managed to acquire one of the first sets of Moretrench equipment to enter Britain and used it on sewer diversion work (Figure 2.3). Harding and his colleagues became experts in the method and contributed to the development of British alternatives to the American equipment. One unexpected result of Harding's expertise was that, during the Second World War, Harding often assisted Royal Engineer bomb disposal units with dewatering for excavations in the search for unexploded bombs (Harding 1981).

In the early 1930s, Harding was also instrumental in the introduction of the modern deep well method to Britain. Mowlem, with Edmund Nuttall Sons & Company, was awarded the contract to construct the King George V graving dock at Southampton to accommodate the liner *Queen Mary*, which was being built on Clydeside. The dock was to be 100 ft (31 m) deep. The Docks Engineer at Southampton was, according to Harding (1981), "A wise and experienced man, he carried out his site investigation to unusual depths." This revealed beds of Bracklesham sands containing water under an artesian head that would reach to above ground level.

At the time, large deep wells using the recently developed submersible pump had been used for groundwater lowering in Germany from 1896 onward, initially for the construction of the Berlin U-Bahn underground railway, but were not a method recognized in Britain. In 1932, Mowlem had obtained licensing agreements with Siemens Bau-Union to use their patents for, among other methods, groundwater lowering by deep wells, with Harding as the nominated British expert. This method was used at Southampton, and groundwater levels in the deep artesian aquifer were successfully lowered by ten deep wells, each equipped with a Siemens

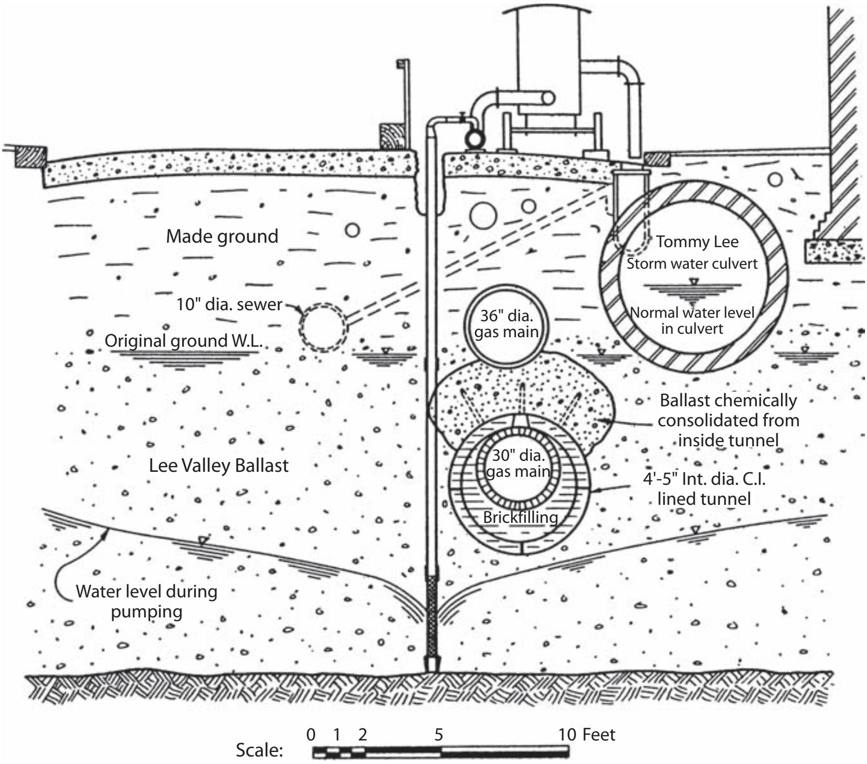


Figure 2.3 Sewer diversion under a gas main using wellpoints and chemical injections. This shows an early British wellpoint application on the Bow–Leyton extension of the London Underground central line in the 1930s. The dewatering allowed tunnel works beneath an existing gas main. (From Harding, H.J.B., *Tunnelling History and My Own Involvement*. Golder Associates, Toronto, Canada, 1981. Courtesy of Golder Associates.)

submersible pump, run from a central control (McHaffie 1938; Harding 1938). This was probably the first rational application of the deep well method in Britain since that by Robert Stephenson for the construction of the Kilsby Tunnel a century earlier.

Further applications of deep well method with submersible pumps followed in the 1940s (Figure 2.4), now unencumbered by license agreements, and the method became an established technique for the control of groundwater (Harding 1946; Glossop and Collingridge 1948). The pioneering practical dewatering work on the Mowlem contracts was continued by Mowlem's subsidiary company, Soil Mechanics Limited, whose groundwater lowering department carried out numerous large-scale projects on power stations and docks in the 1950s and 1960s.

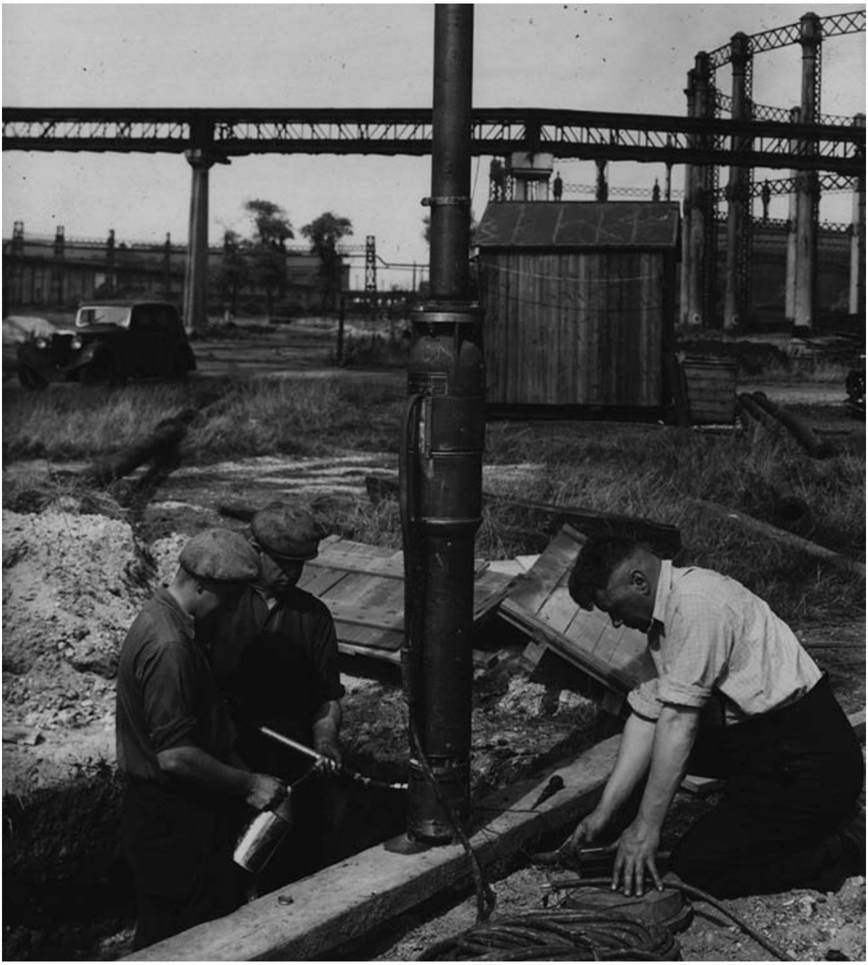


Figure 2.4 Early application of deep wells in Britain. A submersible pump is being prepared for installation into a well on-site in the 1940s.

One of the most recent techniques to be introduced into the United Kingdom is the ejector dewatering method. Jet pumps, which are the basis of the ejector method, were first proposed in the 1850s by Thomson (1852) for the removal of water from water wheel sumps. Dewatering systems using ejectors were developed in the United States almost a century later based on jet pumps used in water supply wells (Prugh 1960; Werblin 1960). The ejector method does not seem to have been much used in the United Kingdom until a few decades after its introduction in North America, although a small-scale ejector system was used in England in 1962 to dewater the Elm

Park Colliery drift (Greenwood 1989). During the 1980s, British engineers and contractors used ejectors on projects in Asia, such as the HSBC headquarters in Hong Kong (Humpheson et al. 1986) and at the Benutan Dam in Brunei (Cole et al. 1994). However, it was not until the late 1980s that a large-scale ejector system was used in the United Kingdom for the casting basin and cut and cover sections of the River Conwy Crossing project in North Wales (Powrie and Roberts 1990).

2.8 PRACTICAL PUBLICATIONS

As described in Chapter 1, the control of groundwater is a practical problem in which theory is only part of the picture: how the theory is put into practice is vital. Historically, the best practical guidance came from in-house dewatering manuals produced by companies such as Geho Pumpen in Holland or the Moretrench American Corporation in North America. One of the first widely published, more practical dewatering texts was the work of Mansur and Kaufman (1962), which formed a chapter of the book *Foundation Engineering* edited by G. A. Leonards. This is a detailed statement-of-the-art of the time, with a strong bias toward the analytical but with some reference to practical considerations. Although it may seem a little dated, this book is essential reading for all who aspire to be specialist dewatering practitioners and wish to understand some of the more accessible theory.

The role of practical engineers in the development of groundwater control technology cannot be overstated. Because many of the techniques were developed from a practical rather than a theoretical basis, much of the experience became concentrated in specialist contracting companies instead of consulting engineers or academic bodies. In the United Kingdom, companies such as Soil Mechanics Limited carried out numerous large-scale groundwater control projects in Britain in the 1950s and 1960s, and many of their staff ultimately moved on and spread their experience around several successful groundwater control contractors in the 1970s, 1980s, and beyond.

The improvements in groundwater control practice were supported by some publications that selflessly shared some of the hard-won experience acquired by contractors. First, in 1981, J. P. Powers, then of the Moretrench American Corporation, produced his book *Construction Dewatering: A Guide to Theory and Practice*, which is understandably oriented toward American practice. Now in its third edition, as Powers et al. (2007), this remains a thorough and readable book. In the United Kingdom, in 1986, the Construction Industry Research and Information Association (CIRIA) produced Report 113 *Control of Groundwater for Temporary Works* (Somerville 1986). It was aimed at the nonspecialist engineering designer and site staff and was largely based on the experience of Pat Cashman from his work with Soil Mechanics Limited, Sykes, and other organizations. In the late 1990s, CIRIA produced another more detailed report. This was Report C515 *Groundwater Control Design and Practice* (Preene et al. 2000), again based on the experience of a specialist contractor, this time, WJ Groundwater Limited. The more recent book by Woodward (2005) also provides some useful information on groundwater control techniques in the context of the range of available ground treatment technologies.

Useful information and case histories can sometimes be found in groundwater conference proceedings. In the 1980s and 1990s, relevant conferences were held on *Groundwater in Engineering Geology* (Cripps et al. 1986), *Groundwater Effects in Geotechnical Engineering* (Hanrahan et al. 1987), and *Groundwater Problems in Urban Areas* (Wilkinson 1994).

This book is intended to complement and augment these texts and will concentrate, in the main, on the practical requirements for groundwater control for temporary works.

Chapter 3

Groundwater and permeability

3.1 INTRODUCTION

To allow even a basic approach to the control of groundwater, the practitioner should be familiar with some of the principles governing groundwater flow. Similarly, the specialist terms and language used must be understood. This chapter briefly describes the circumstances behind the flow of water through the ground. Chapter 4 will describe how groundwater flow affects the stability of construction excavations.

This chapter outlines the hydrological cycle and introduces the concepts of aquifers and permeability (a measure of how easily water can flow through a porous mass). Darcy's law—which is used to describe most groundwater flow regimes—is described, and typical values of permeability are presented (the problems of obtaining meaningful permeability values will be covered in Chapter 6). Because this is a practical text, our main interest is in abstracting water from the ground with wells or sumps of one kind or another. Accordingly, the principles of groundwater flow to wells are presented.

The importance of aquifer types and geological structure, both in the large and small scale, on groundwater flow is discussed. Two geological case histories are presented. Finally, this chapter discusses basic groundwater chemistry, and its relation to groundwater lowering problems.

3.2 HYDROLOGY AND HYDROGEOLOGY

The study of water, and its occurrence in all its natural forms, is called "hydrology." This branch of science deals with water, its properties, and all of its behavioral phases. It embraces geology, soil mechanics, meteorology, and climatology, as well as hydraulics and the chemistry and bacteriology of water.

Terminology may vary from country to country but, in general, those professionals who specialize in the hydrology of groundwater are known as "hydrogeologists." The study and practice of hydrogeology is of increasing importance both in developed and in developing countries where groundwater is used as a resource, to supply water for the population and its industries; for example, around one-third of the United Kingdom's drinking water is obtained from groundwater. The reader interested in hydrogeology is commended to an introductory text by Price (1996), and more theoretical treatments by Fetter (1994) or Younger (2007) among the copious literature on the subject.

For the dewatering practitioner, detailed knowledge of some of the more arcane areas of hydrogeology is not necessary. However, if a rational approach is to be adopted, it is vital that the basic tenets of groundwater flow are understood in principle. Further study, beyond basic principles, will require some understanding of higher mathematics and analysis—this can be very useful, but will not be for everyone. The references given at the end of this book should allow interested and motivated readers to pursue the subject as far as they wish.

It is worth mentioning that there is sometimes a little professional friction between dewatering practitioners (generally from a civil engineering background) and hydrogeologists (often with a background in earth sciences). For their part, the dewatering engineers are often pragmatic. They have an unstable excavation and need to control groundwater accordingly—they see it very much as a local problem. Hydrogeologists, on the other hand, are trained to view groundwater as part of the wider environment. They see that a groundwater lowering system should not be considered in isolation—but sometimes they may not fully appreciate the practical limits of available dewatering methods. Like many professional rivalries, the wise observer can learn from both camps and gain a more rounded view of the problem in hand. This is the approach that the authors wish to encourage. Some of the misunderstandings between dewatering practitioners and hydrogeologists have resulted from the slightly different terminologies used by each group. This book unashamedly prefers the engineering terms used in dewatering practice, but introduces and explains alternative forms where appropriate.

3.2.1 The hydrological cycle

The hydrological cycle is now so widely accepted that it is difficult to conceive of any other concept. The hydrological cycle is based on the premise that the volume of water on earth is large, but finite. Most of this water is in continuous circulation. Water vapor is taken up into the atmosphere from surface water masses (principally the oceans) to form clouds; later, cold temperatures aloft cause the water to fall as precipitation (dew, fog, rain, hail, or snow) on the earth's surface from whence it is eventually returned to the oceans (Figure 3.1), from where the cycle is repeated.

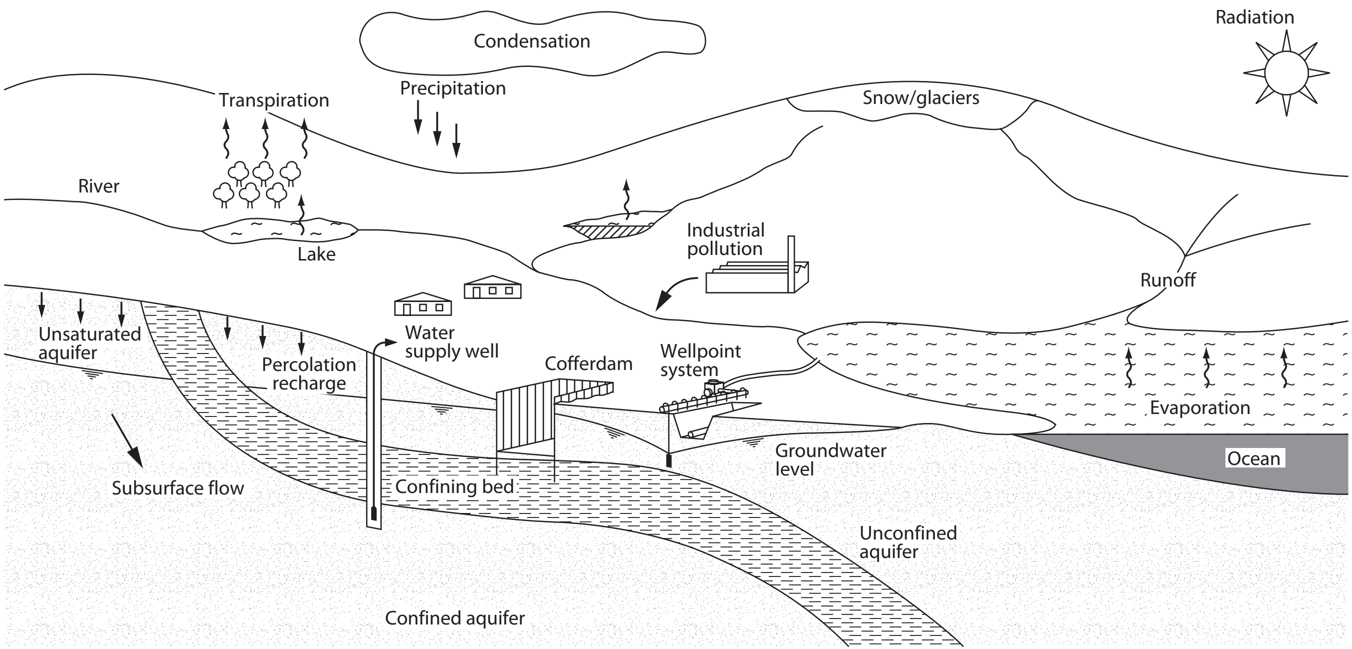


Figure 3.1 The hydrological cycle. (From Preene, M., Roberts, T.O.L., Powrie, W., and Dyer, M.R. Groundwater control—Design and practice. *CIRIA Report C515*, Construction Industry Research and Information Association, London, 2000. Reproduced by kind permission of CIRIA: www.ciria.org.)

Soil and rock are made up of mineral particles in contact with each other. The soil and rock masses contain voids, either widely distributed in the form of pores, or locally concentrated as fractures or fissures. In simple terms, the water contained in the voids of the soil and rock is known as groundwater.

Generally, a "water table" will exist at some depth below the ground's surface; below the water table, the soil and rock pores and fissures are full of water (and are said to be saturated). Above the water table, the pores and fissures are unsaturated (i.e., they contain both water and air). The hydrological cycle illustrates clearly that precipitation replenishes the groundwater in the voids of the soils and rocks and that gravity flow causes the movement of groundwater through the pores or fissures of soil and rock masses, toward rivers and lakes, and eventually, to the oceans.

It is unlikely that all of the precipitation that falls on landmasses will percolate downward sufficiently far enough to replenish groundwater.

1. Some precipitation may fall onto trees or plants and evaporate before ever reaching the ground.
2. Some precipitation will contribute to surface runoff. This proportion will depend mainly on the composition of the ground surface. For instance, an urban paved surface area will result in almost total surface runoff, with little percolation into the ground.
3. Of that precipitation which does enter the ground, not all of it will percolate sufficiently far enough to reach the water table. Some will be taken up by plant roots in the unsaturated zone just below the ground surface and will be lost as evapotranspiration from the plants.

One unavoidable conclusion from the hydrological cycle is that most groundwater is continually in motion, flowing from one area to another. Under natural conditions (i.e., without interference by man in the form of pumping from wells), groundwater flow is generally relatively slow, with typical velocities being in the range of a few meters per day in high-permeability soils and rocks down to a few millimeters per year in very low-permeability deposits. Larger groundwater velocities generally only exist in the vicinity of pumped wells or sumps.

The stability of the hydrological cycle can be illustrated by considering groundwater temperature. Price (1996) states that in temperate climates the temperature of groundwater below a few meters' depth tends not to vary with the seasons and remains remarkably constant at around the mean annual air temperature. In the United Kingdom, this is around 10°C to 14°C. This is obvious on site if you have to take a water sample from the pumped discharge of a dewatering system. On a frosty winter morning, the pumped groundwater will seem pleasantly warm on your hands, but take a similar sample on a warm summer day and the discharge water will seem icy cold. At greater depths in bedrock, the temperature stays constant with time, but increases with depth. This is known as a geothermal temperature gradient, mainly the result of heat generated by the decay of radioactive materials in the rock.

The relatively constant year-round temperature of the ground and groundwater at a site can be harnessed via ground energy systems to provide heating and cooling for buildings or industrial processes. Using either open- or closed-loop ground collectors (Figure 3.2), heat energy can be exchanged between a building and the ground. This heat energy is then

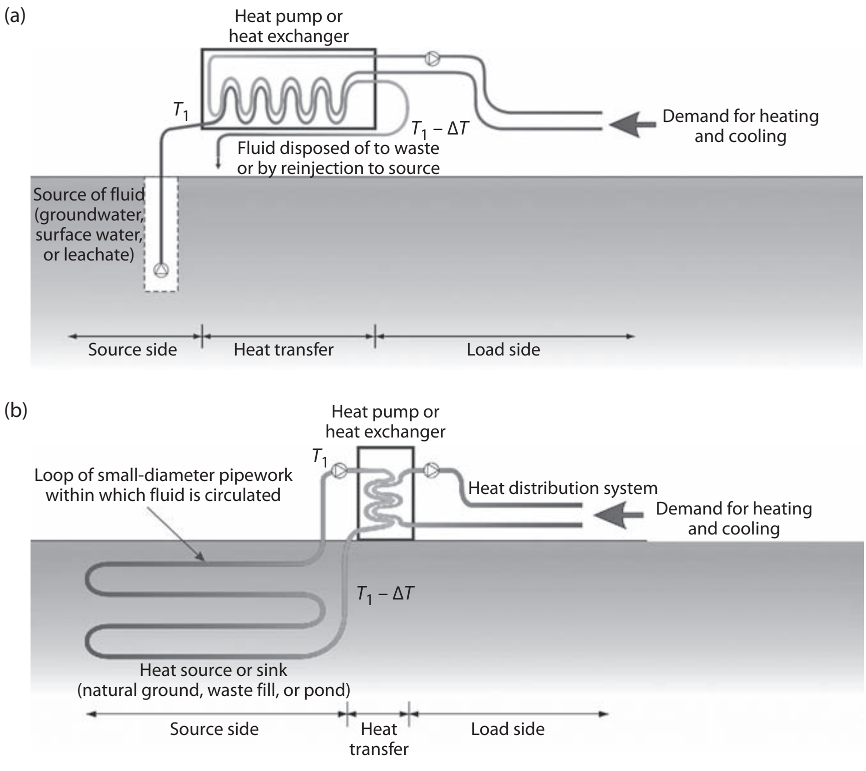


Figure 3.2 Ground energy system. (a) Open-loop system. Groundwater is abstracted from the source (typically one or more boreholes), passed through a heat pump or heat exchanger, and disposed of to either waste (sewer or watercourse) or by reinjection to the source (typically by one or more aquifer reinjection boreholes). (b) Closed-loop system. A thermal transfer fluid is circulated through a closed circuit of pipework embedded in the ground, thereby allowing the building heat pump system to reject or extract heat from the ground. The ground loop can be configured into shallow trenches, an array of vertical boreholes, or incorporated into the building piles and other foundations. (From Preene, M. and Powrie, W. *Proceedings of the Institution of Civil Engineers*, *Energy*, 162, 77–84, 2009. Reproduced by permission of ICE Publishing.)

used in place of traditional heating and cooling systems (such as gas-fired boilers or electrically powered cooling systems), thereby reducing carbon dioxide emissions from buildings. Further details of ground energy systems (also known as geothermal systems) are given in Banks (2008) and Preene and Powrie (2009).

3.2.2 Geology and soil mechanics

The successful design and application of groundwater lowering methods depends not just on the nature of the groundwater environment (such as where the site is within the hydrological cycle) but also on the critical influence of the geology or structure of the soils and rocks through which water flows. This is especially true when trying to assess the effect of groundwater conditions on the stability of engineering excavations (as will be discussed in Chapter 4). The eminent soil mechanics engineer, Karl Terzaghi wrote in 1945: "It is more than mere coincidence that most failures have been due to the unanticipated action of water, because the behavior of water depends, more than on anything else, on minor geological details that are unknown" (Peck 1969). This is still true today, but perhaps the last sentence should to be changed to "minor geological details that are often overlooked during site investigation"— good practice for site investigation will be discussed in Chapter 6.

The reader wishing to apply groundwater lowering methods will need to be familiar with some of the aspects of hydrogeology, soil mechanics (including its applied form, geotechnical engineering), and engineering geology; texts by Powrie (2004) and Blyth and De Freitas (1984), respectively, are recommended as introductions to the latter two subjects. However, as has been stated earlier, study of theory is only part of the learning process. Many soils and rocks may not match the idealized homogeneous isotropic conditions assumed for some analytical methods. Judgment will be needed to determine how credible the results are of analyses based on such models.

An important geological distinction is made between uncemented "drift" deposits (termed "soil" by engineers) and cemented "rock." Drift deposits are present near the ground surface and consist of sand, gravel, clay and silt, which may have resulted from weathering or from glacial or alluvial processes. Groundwater flow through drift deposits is predominantly intergranular—that is, through the pores in the mass of the soil. Rock may be exposed at the ground surface or may be covered by layers of drift and could consist of any of the many rock types that exist, such as sandstone, mudstone, limestone, basalt, gneiss, and so on. Groundwater flow through rock is often not through pores (which may be too small to allow significant passage of water) but along fissures, fractures, and joints within the rock mass. This means that groundwater flow through rock may be concentrated locally, where the deposition and subsequent solution or tectonic action has created fissure networks.

Because of their uncemented nature, drift deposits could cause severe stability problems for excavations below the groundwater level. Most groundwater lowering operations are carried out in drift, and this book will concentrate on those situations. In some circumstances groundwater lowering is also carried out in rock (see Section 4.8).

3.3 PERMEABILITY AND GROUNDWATER FLOW

Permeability is a critical parameter for the assessment of how water flows through soil and rocks. The precise meaning of the term "permeability" is sometimes a cause of confusion between engineers and hydrogeologists. Civil and geotechnical engineers are interested almost exclusively in the flow of water through soils and rocks and use the term "coefficient of permeability," given the symbol k. For convenience, k is generally referred to simply as "permeability," and throughout this book, this terminology will be used. Therefore, for groundwater lowering purposes, permeability, k, will be defined as "a measure of the ease or otherwise with which groundwater can flow through the pores of a given soil mass."

A slight complication is that the permeability of porous mass is dependent not only on the nature of the porous media, but also on the properties of the permeating fluid. In other words, the permeability of a soil to water is different from the permeability of the soil to another fluid, such as air or oil. Hydrogeology references highlight this by calling the engineer's permeability "hydraulic conductivity" to show that it is specific to water. Often, if the term permeability appears in hydrogeology references, it actually means the permeability of the porous media independent of the permeating fluid, sometimes also known as intrinsic permeability (see Price 1996, p. 52).

3.3.1 Darcy's law

The modern understanding of the flow of groundwater through permeable ground originates with the researches of the French hydraulics engineer, Henri Darcy (1856; see Section 2.4). He investigated the purification of water by filtration through sand and developed an equation of flow through a granular medium based on the earlier work of Pouseuille concerning flow in capillary tubes. His conclusions can be expressed algebraically as Equation 3.1, universally referred to as Darcy's law, which forms the basis for most methods of analysis of groundwater flow.

$$Q = kA\left(\frac{dh}{l}\right) \tag{3.1}$$

where (see Figure 3.3, which schematically shows Darcy's experiment):

Q = the volumetric flow of water per unit time (the "flow rate")
A = the cross-sectional area through which the water flows
l = the length of the flow path between the upstream and downstream ends
dh = the difference in total hydraulic head between the upstream and the downstream ends
k = the permeability of the porous media through which the water flows

The total hydraulic head at a given point is the sum of the "pressure head" and the "elevation head" at that point, as shown on Figure 3.4. The elevation head is the height of the measuring point above an arbitrary datum, and the pressure head is the pore water pressure u, expressed as meters of head of water. Total hydraulic head is important because it controls groundwater flow. Water will flow from high total hydraulic head to low total hydraulic head. It follows that water does not necessarily flow from high pressure to low pressure or from high elevations to low elevations—it will only flow in response to total hydraulic head, not pressure or elevation considered in isolation.

A term which is used frequently in any discussion of groundwater lowering is "drawdown." Drawdown is the amount of lowering (in response to pumping) of total hydraulic head, and is a key, measurable, performance target for any dewatering operation. Drawdown is equivalent to

1. The amount of lowering of the "water table" in an unconfined aquifer.
2. The amount of lowering of the "piezometric level" in a confined aquifer.

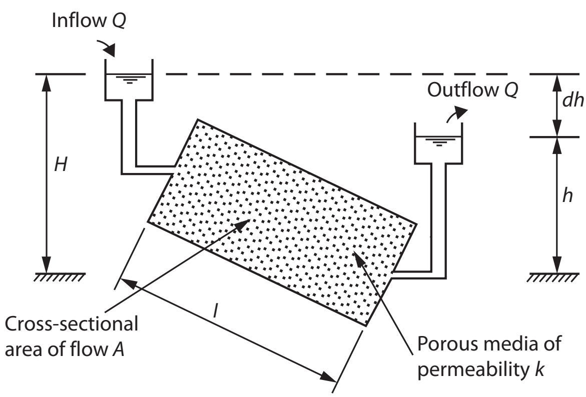


Figure 3.3 Darcy's experiment.

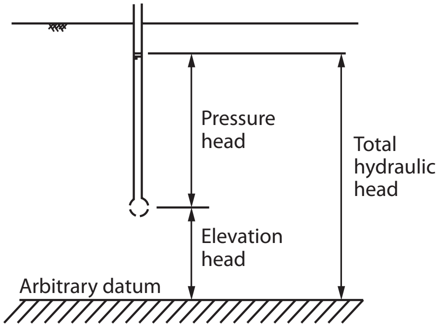


Figure 3.4 Definition of a hydraulic head.

3. The reduction (expressed as meters of head of water) of pore water pressure observed in a piezometer (see Section 6.6). In this case, the drawdown of total hydraulic head can be estimated directly from the change in the pressure head because the level of the piezometer tip does not change, so the change in elevation head is zero. If the reduction in pore water pressure is Δu, the drawdown is equal to $\Delta u/\gamma_w$, where γ_w is the unit weight of water.

Darcy's law is often written in terms of the hydraulic gradient i which is the change in hydraulic head divided by the length of the flow path ($i = dh/l$). Equation 3.1 then becomes

$$Q = kiA \tag{3.2}$$

In this form, the key factors affecting groundwater flow are obvious

1. If other factors are equal, an increase in permeability will increase the flow rate.
2. If other factors are equal, an increase in the cross-sectional area of flow will increase the flow rate.
3. If other factors are equal, an increase in hydraulic gradient will increase the flow rate.

These points are vital in beginning to understand how groundwater can be manipulated by groundwater lowering systems.

In the presentation of his equation, Darcy left no doubt of its origin being empirical. His important contributions to scientific knowledge were based on careful observation in the field and in the laboratory, and on the conclusions that he drew from these. Permeability is in fact only a theoretical concept, but one vital to realistic assessments of groundwater pumping requirements and so an understanding of it is most desirable. In theory, permeability is the notional (or "Darcy") velocity of flow of pore water through unit cross-sectional area. In fact, the majority of the cross-sectional area of a soil mass actually consists of soil particles through which pore water cannot flow. The actual pore water flow velocity is greater than the "Darcy velocity," and is related by the soil porosity n (porosity is the ratio of voids, or pore space, to total volume).

The main condition for Darcy's law to be valid is that groundwater flow should be "laminar," a technical term meaning that the flow is smooth. Flow will be laminar at low velocities but will become turbulent above a certain velocity, dependent on the porous media and the permeating fluid. Darcy's law is not valid for turbulent flow. In most groundwater lowering applications, flow can safely be assumed to be laminar. The only location where turbulent flow is likely to be generated is close to high flow rate wells pumping from coarse gravel aquifers. The implications of this for flow to wells are discussed in Section 3.5.

For idealized and homogeneous soils, permeability depends primarily on the properties of the soil, including the size and arrangement of the soil particles, and the resulting pore spaces formed when the particles are in contact. For example, consider an assemblage of billiard balls of similar sizes (Figure 3.5a). This is analogous to the structure of a high-permeability soil (such as a coarse gravel) in which the voids (or pore water passages) are large, and the pore water can flow freely. Next, consider an assemblage of billiard balls with marbles placed in the spaces between the billiard balls (Figure 3.5b); this is analogous to a soil of moderate permeability because the effective size of the pore water passages are reduced. Finally, consider a structure with lead shot particles placed in the voids between the billiard balls and the marbles—the passages for the flow of pore water are further reduced; this simulates a low-permeability soil (Figure 3.5c). It is logical to infer from this analogy that the "finer" portion of a sample dominates permeability. The coarser particles are just the skeleton of the soil and may have little bearing on the permeability of a sample or of a soil mass in situ.

The proportion of different particle sizes in a sample of soil (as might be recovered from a borehole) can be determined by carrying out a particle size distribution (PSD) test (Head 1982). The results of the test are normally presented as PSD curves, such as Figure 3.6, which shows typical PSD curves for a range of soils. PSD curves are sometimes known as grading curves or sieve analyses, after the sieving methods used to categorize the coarser particle sizes. Because there is a fairly intuitive link between particle size and permeability, over the last hundred years or so many researchers and practitioners have developed empirical correlations between permeability and certain particle size characteristics. Some of these methods are still in use today, and are described in Section 6.7, along with the drawbacks and limitations of such correlations.

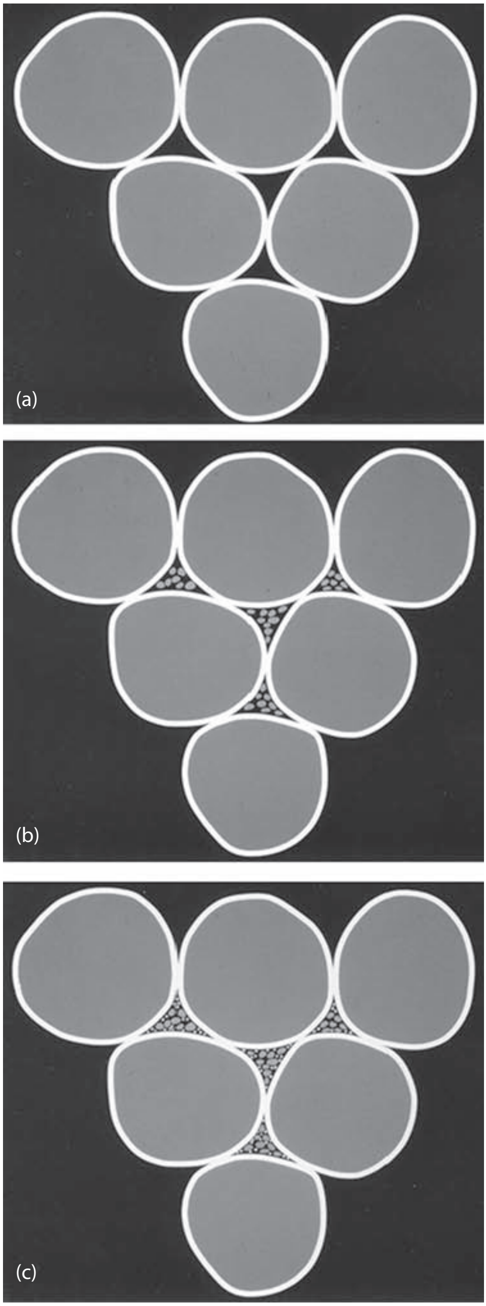


Figure 3.5 Soil structure and permeability. (a) High permeability. (b) Medium permeability. (c) Low permeability.

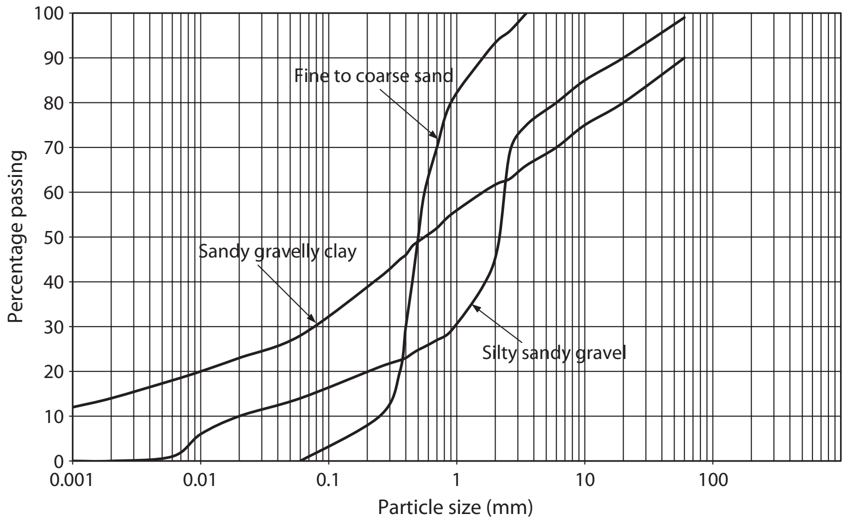


Figure 3.6 Particle-size distribution.

As discussed earlier, the permeability is also dependent on the properties of the permeating fluid. Groundwater lowering is concerned exclusively with the flow of water, so this issue is not a major concern. It is worth noting, however, that the viscosity of water varies with temperature (and varies by a factor of two for temperatures between 20°C and 60°C); therefore, in theory, permeability will change with temperature. In practice, groundwater temperatures in temperate climates vary little and errors in permeability due to temperature effects tend to be small in comparison with other uncertainties.

3.3.2 Typical values of permeability

In practice, meaningful values of permeability are more difficult to visualize than in Darcy's experiment. Darcy's concept of permeability, described previously, assumes that the soil permeability is homogenous (i.e., it is the same everywhere) and isotropic (i.e., it has the same properties in all directions). Even a basic study of geology will show that many soils and rocks exhibit properties far removed from these assumptions. Many strata are more or less heterogeneous in character, so that even in an apparently consistent stratum, the permeability may vary greatly between one part and another. This may result from inhomogeneities such as fissures, erosion features, or sand and clay lenses. Also, the nature of deposition of soils can introduce anisotropy, in which the soil permeability is not the same in all directions. Soils laid down in water may have a layered or laminated structure or "fabric"—as a result, the permeability in a horizontal direction is usually significantly greater than that in the vertical direction. The influence of soil fabric on permeability is discussed in detail by Rowe (1972).

Despite these complications, any rational attempt at groundwater lowering will require permeability to be investigated and assessed. Some of the design methods discussed in Chapter 7 assume, for simplicity, isotropic and homogeneous permeability conditions—on a theoretical basis, this is clearly unrealistic. Yet these methods are established and field-proven methods and, if applied appropriately, can give useful results. This highlights the principle that in ground engineering, it is often necessary to tolerate some simplification to make a problem more amenable to analysis. The key is to apply critical judgment to the parameters used in analysis, and not to blindly accept the results of the analysis until they are corroborated or validated by other data or experience.

As will be described in the section on site investigation (Chapter 6), tests to estimate realistic values of permeability can be problematic. As an engineering parameter, permeability is unusual because of the tremendous range of possible values for natural soils and rocks; there can be a factor of perhaps 10^{10} between the most permeable gravels and almost impermeable intact clays. It is not unknown for the results of permeability tests to be in error by a factor of 10, 100, 1000, or even more. This often stems purely from the limitations of the test and interpretation methods themselves, and can occur even if the tests are carried out in an exemplary fashion by experienced personnel.

Table 3.1 provides some general guidance on the permeability of typical soil types. This table is intended to be used for comparison purposes with permeability test results, to look for inconsistencies between soil descriptions (from borehole logs) and reported test results. Nevertheless, this table should be used with caution, especially in mixed soil types, in which the soil fabric (which is sometimes not adequately indicated in the soil description) may play a dominant role.

Table 3.1 Typical values of soil permeability

Soil type	*Typical classification of permeability*	*Permeability (m/s)*
Clean gravels	High	$>1 \times 10^{-3}$
Clean sand and sand/gravel mixtures	High to moderate	1×10^{-3} to 5×10^{-4}
Fine and medium sands	Moderate to low	5×10^{-4} to 1×10^{-4}
Silty sands	Low	1×10^{-4} to 1×10^{-6}
Sandy silts, very silty fine sands, and laminated or mixed strata of silt/sand/clay	Low to very low	1×10^{-5} to 1×10^{-8}
Fissured or laminated clays	Very low	1×10^{-7} to 1×10^{-9}
Intact clays	Practically impermeable	$<1 \times 10^{-9}$

The permeability values used throughout this book will be reported in meters per second (m/s), which is the convention in engineering and soil mechanics. This is in contrast with hydrogeological references, which generally report permeability (or hydraulic conductivity) in meters per day (m/d). Conversion factors between various units are given at the end of this book.

3.4 AQUIFERS, AQUITARDS, AND AQUICLUDES

"Aquifer" is a useful term that appears whenever groundwater is discussed. As used by hydrogeologists, an aquifer might be defined as "a stratum of soil or rock which can yield groundwater in economic or productive quantities." Almost all wells used for water supply purposes are drilled into, and pump from, aquifers. Examples of aquifers in the United Kingdom include the Chalk or Sherwood Sandstone. By this definition, strata which yield water at flow rates too small to be used for supply are not aquifers, and might be considered "nonaquifers." Examples of nonaquifers might include alluvial silts, glacial lake deposits, or fissured mudstones.

From a groundwater lowering point of view, the hydrogeologists' definition of aquifer in terms of "productive quantities" is unhelpful. The groundwater in many strata that yield just a little water (and so are nonaquifers) can cause severe problems for excavation stability (see Chapter 4). For the purposes of this book, the definitions of Construction Industry Research and Information Association (CIRIA) Report C515 (Preene et al. 2000) will be adopted, in which the reference to productive quantities is omitted. These definitions are given below, together with those for aquiclude and aquitard. The relationship between these strata types is discussed in the following sections.

Aquifer. Soil or rock forming a stratum, group of strata, or part or stratum that is water-bearing (i.e., saturated and permeable).

Aquiclude. Soil or rock forming a stratum, group of strata, or part or stratum of very low permeability, which acts as a barrier to groundwater flow.

Aquitard. Soil or rock forming a stratum, group of strata, or part or stratum of intermediate to low permeability, which yields only very small groundwater flows.

3.4.1 Unconfined aquifers

An unconfined aquifer is probably the simplest for most people to visualize and understand. The soil or rock of the aquifer contains voids, pores, or fissures. These voids are saturated (i.e., full of groundwater) up to a certain level known colloquially as the "water table" (Figure 3.7a), which is open to the atmosphere. An observation well (see Section 6.6), installed into the saturated part of the aquifer, will show a water level equivalent to the water table. The analogy that can easily be drawn is that of digging a hole in the sand on a beach. Water does not enter the hole until the water table is reached, at which point water will enter the hole and stay at that level unless water is pumped out.

The water table can be defined as level in the aquifer at which the pore water pressure is zero (i.e., equal to atmospheric). Below the water table, the soil voids are at positive pore water pressures and are saturated. Above the water table, the pressure in the voids will be negative (i.e., less than atmospheric) and, depending on the nature of the soil or rock, may be unsaturated and contain both water and air. In most unconfined aquifers, the great majority of groundwater flow occurs below the water table. When the

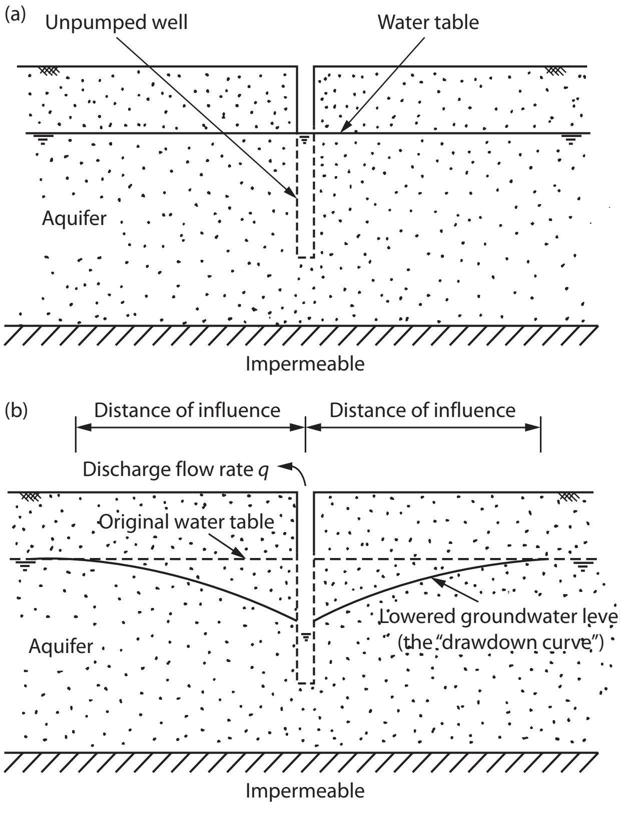


Figure 3.7 Unconfined aquifer. (a) Unpumped conditions. (b) During pumping.

water table is lowered by pumping, the thickness of the saturated aquifer will be reduced.

If groundwater is pumped (or abstracted) from an unconfined aquifer, it is intuitively apparent that the water table will be lowered locally around the well and a "drawdown curve" will be created (Figure 3.7b). In simple terms, the drawdown curve is the new, curved shape of the water table. Pore water will drain out of the soil above the new lowered water table, and will be replaced by air—this soil will become unsaturated. The amount of water contained in a soil will depend on the porosity of the soil, but it is important to note that not all of the water in an unconfined aquifer will drain out when the water table is lowered. Some water will be retained in the smaller soil pores by capillary forces. The proportion of water which can drain from an unconfined aquifer is described by the specific yield S_y, which is generally less than the soil porosity.

The zone above the water table (sometimes known as the vadose zone) is worthy of further consideration. Generally, the soil in this zone above the standing water table will be moist due to the presence of capillary water in the interconnected pores between soil particles. This capillary water differs from groundwater below the water table in that it is hardly noticeable in a borehole or excavation because it is below atmospheric pressure and cannot flow into the excavation. It is held in place by capillary tension, at negative pore water pressures (Figure 3.8).

The soil is saturated with capillary water for a height above the water table (this is the capillary saturated zone). The height of the capillary

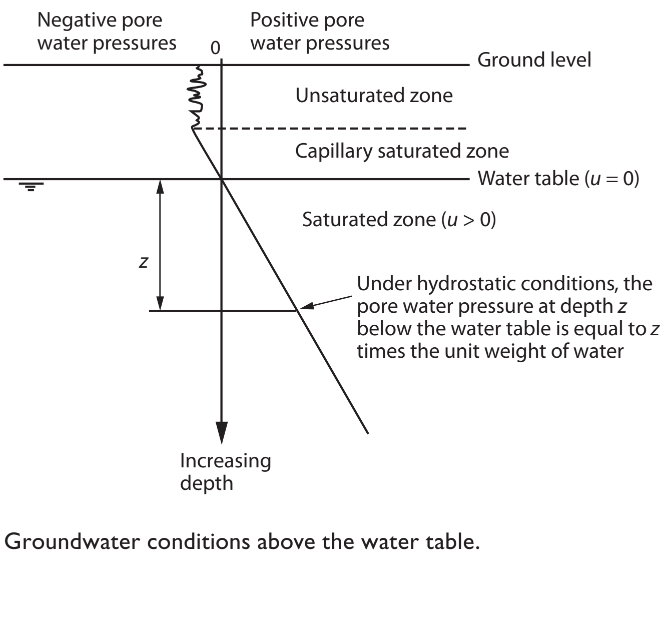


Figure 3.8 Groundwater conditions above the water table.

saturated zone above the water table is dependent on the effective size of the pores—the smaller the pores, the greater the height. At the upper surface of the capillary saturated zone, the surface tension forces between the water molecules and the soil particles are only just sufficient to prevent air from being drawn into the soil pores. Above the capillary saturated zone, air can enter the soil, which becomes unsaturated. Typical heights of the capillary zone might be a few centimeters for coarse sand, or several meters for finer-grained soils (such as silts and clays) where the effective pore size is smaller.

3.4.2 Confined aquifers

The distinction between unconfined and confined aquifers is important because they behave in quite different ways when pumped. In contrast to an unconfined aquifer, in which the top of the aquifer is open to the atmosphere and an unsaturated zone may exist above the water table, a confined aquifer is overlain by a very low-permeability layer known as an "aquiclude" which forms a confining bed. A confined aquifer is saturated throughout because the water pressure everywhere in the aquifer is above atmospheric. An observation well drilled into the aquifer would initially be dry when drilled through the confining bed. When the borehole penetrates the aquifer, water will enter the borehole and rise to a level above the top of the aquifer. Because the pore water pressures everywhere are above atmospheric, a confined aquifer does not have a water table. Instead, its pressure distribution is described in terms of the "piezometric level," which represents the height to which water levels will rise in observation wells installed into the aquifer (Figure 3.9a).

If a confined aquifer is pumped, the piezometric level will be lowered to form a drawdown curve, which represents the new, lower, pressure distribution in response to pumping (Figure 3.9b). Provided that the piezometric level is not drawn down below the top of the aquifer (i.e., the base of the confining bed), the aquifer will remain saturated. Water will not drain out of the soil pores to be replaced by air in the manner of an unconfined aquifer. Instead, a confined aquifer yields water by compression of the aquifer structure (reducing pore space) and expansion of the pore water in response to the pressure reduction. The proportion that can be released from a confined aquifer is described by the storage coefficient S.

If the water pressure in the confined aquifer is sufficiently high, a well drilled through the confining bed into the aquifer will be able to overflow naturally at ground level and yield water without pumping (Figure 3.10). This is known as the flowing artesian condition—artesian is named after the Artois region of France, where such conditions were first recorded. Flowing artesian conditions are possible from wells drilled in low-lying areas into a

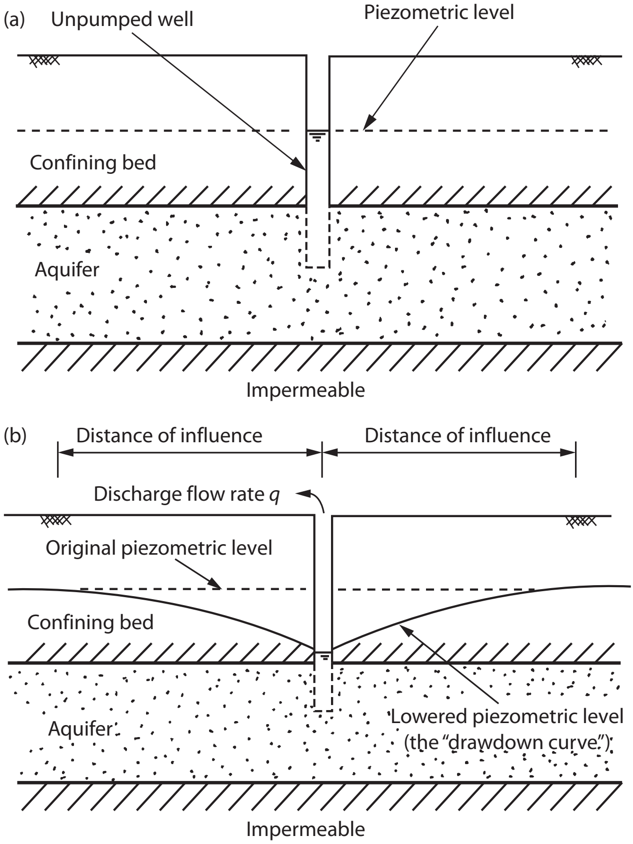


Figure 3.9 Confined aquifer. (a) Unpumped conditions. (b) During pumping.

confined aquifer that is recharged from surrounding high ground. Flowing artesian aquifers are a special case of confined aquifers—confined aquifers that do not exhibit flowing conditions are sometimes known as artesian aquifers or occasionally (and incorrectly) as subartesian aquifers.

3.4.3 Aquicludes

An aquiclude is a very low-permeability layer that will effectively act as a barrier to groundwater flow, for example, as a confining bed above an aquifer. It need not be completely "impermeable" but should be of sufficiently low permeability that during the life of the pumping system, only negligible amounts of groundwater will flow through it. The most common forms of aquiclude are layers of relatively unfissured clay or rock with permeabilities of 10^{-9} m/s or less.

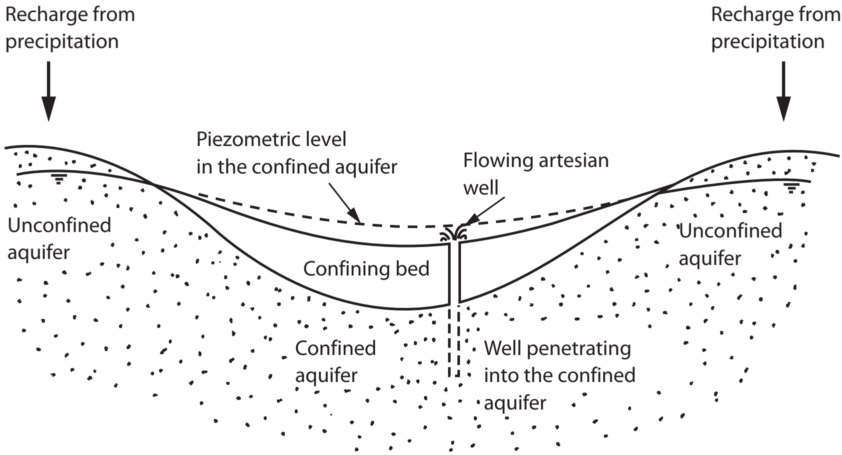


Figure 3.10 Flowing artesian conditions.

In addition to having a low permeability, a stratum should meet two other criteria before it can be considered to act as an aquiclude:

1. It must be continuous across the area affected by pumping; otherwise, water may be able to bypass the aquiclude.
2. It must be of significant thickness. A thin layer of extremely low-permeability material may be less effective as an aquiclude compared with a much thicker layer of greater, but still low, permeability. The thicker a layer of clay or rock is, the more likely it may act as an aquiclude.

3.4.4 Aquitards and leaky aquifers

An aquitard is a stratum of intermediate permeability between an aquifer and an aquiclude. In other words, it is of sufficiently low permeability that it is unlikely that anyone would consider installing a well to yield water, but neither is it of such low permeability that it can be considered effectively impermeable. Soil types that may form aquitards include silts, laminated clays/sands/silts, and certain clays and rocks that, although relatively impermeable in themselves, contain a more permeable fabric of fissures or laminations.

Aquitards are of interest to hydrogeologists because they form part of "leaky aquifer" systems. Such a system (also known as a semiconfined aquifer) consists of a confined aquifer in which the confining layer is not an aquiclude, but is an aquitard (Figure 3.11). When the aquifer is pumped, water will flow vertically downward from the aquitard and "leak" into the aquifer, ultimately contributing to the discharge flow rate from the well.

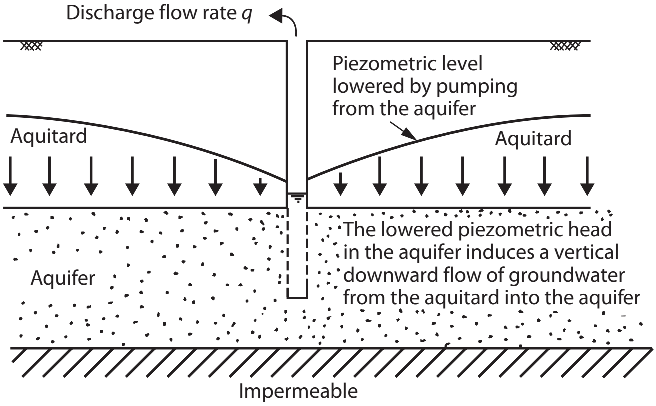


Figure 3.11 Leaky aquifer system.

It is apparent that the term "leaky aquifer" is a misnomer because it is the aquitard that is actually doing the leaking.

Aquitards are relevant to the dewatering practitioner for the following reasons:

1. If aquitards leak into underlying pumped aquifers, the effective stress (see Section 4.3) will increase, leading to consolidation settlements (see Section 15.4). Analysis of the behavior of any aquitards present is important when assessing the risk of damaging settlements.
2. Although aquitards may not yield enough water to form a supply, construction excavations into aquitards are likely to encounter small but problematic seepages and instability problems. Many applications of pore water pressure control systems using some form of vacuum wells (see Section 5.5) are carried out in soils that would be classified as aquitards.

3.4.5 Aquifer parameters

The concept of permeability k was introduced previously (see Section 3.3), and is an important parameter used to describe aquifer properties. The thickness, D, of an aquifer is also important because thicker aquifers of a given permeability will yield more water than thinner ones. These two terms can be combined into the hydrogeological term "transmissivity" (T).

$$T = kD \tag{3.3}$$

For SI units, k and D will be in meters per second and meters, respectively, so T will have units of meters squared per second. The results of pumping tests (see Section 6.7) are sometimes reported in terms of transmissivity; Equation 3.3 allows these to be related to permeability. In unconfined aquifers, where the aquifer thickness is reduced as a result of pumping, transmissivity will be reduced in a similar manner.

The amount of water released from an aquifer as a result of pumping is described by the storage coefficient. This is defined as the volume of water released from storage, per unit area of aquifer, per unit reduction in head; it is a dimensionless ratio. Because of the different ways by which water is yielded by confined and unconfined aquifers, the storage coefficient is dealt with differently for each.

For an unconfined aquifer, the storage coefficient is termed the specific yield, S_y. This indicates how much water will drain out of the soil, to be replaced by air, under the action of gravity. Coarse-grained aquifers, such as sands and gravels, yield water easily from their pores when the water table is lowered. Finer-grained soils, such as silty sands, have smaller pores in which capillary forces may retain much of the pore water even when the water table is lowered; S_y may be much lower than for gravels. Typical values of S_y are given in Table 3.2. Surface tension forces may also mean that the water may not drain out of the pores instantaneously when the water table is lowered; it may drain out slowly with time. This phenomenon is known as "delayed yield" and can affect drawdown responses in unconfined aquifers.

In a confined aquifer, because the aquifer remains saturated, there is no specific yield and the storage coefficient, S, is used to describe the aquifer behavior. Because water is only released by compression of the aquifer and expansion of the pore water, typical values of S will be small, perhaps on the order of 0.0005–0.001. More compressible confined aquifers will yield more water under a given drawdown, and tend to have a greater storage coefficient, compared with stiffer aquifers. If the piezometric level in the aquifer is lowered sufficiently that it falls below the top of the aquifer, unconfined conditions will develop and a value of S_y will apply to the unconfined part of the aquifer.

Table 3.2 Typical values of specific yield

Aquifer	*Specific yield*
Gravel	0.15–0.30
Sand and gravel	0.15–0.25
Sand	0.10–0.30
Chalk	0.01–0.04
Sandstone	0.05–0.15
Limestone	0.005–0.05

Sources: Sterrett, R., *Groundwater and Wells*, 3rd edition, Johnson Division, St. Paul, MN, 2008. With permission. Oakes, D.B. Theory of groundwater flow. *Groundwater: Occurrence, Development and Protection* (Brandon, T.W., ed.), Water Practice Manual No. 5, Institution of Water Engineers and Scientists, London, pp. 109–134, 1986. With permission.

Most of the design methods presented later in this book (see Chapter 7) are based on simple, steady state methods commonly used for temporary works construction applications. The storage coefficient does not appear in those calculations because by the time steady state has occurred, all the water will have been released from storage. For relatively small construction excavations, this is a reasonable assumption because steady state is generally achieved within a few days or weeks, thus the volumes from storage release are only a concern during the initial drawdown period. Storage volumes may be more of a concern for large quarrying or open pit mining projects when steady state may take a much longer time to develop.

3.5 FLOW TO WELLS

Our interest in groundwater is more than purely academic; we need to be able to control groundwater conditions to allow construction to proceed. The principal approach is to install a series of wells and to pump or abstract water from these wells—this is the essence of groundwater lowering or dewatering. Hence, flow toward wells must be understood.

Some definitions will be useful. For the purposes of this book, a "well" is any drilled or jetted device that is designed and constructed to allow water to be pumped or abstracted. These definitions are, again, slightly different from those used by some hydrogeologists. In hydrogeology, a well is often taken to mean a large diameter (i.e., >1.8 m) well or shaft, such as may be dug by hand in developing countries. A smaller diameter well, constructed by a drilling rig, is often termed a "borehole," or in developing countries, a "tube well."

A well will have a well screen (a perforated or slotted section that allows water to enter from certain strata) and a well casing (which, conversely, prevents water from entering from certain strata; see Figure 3.12).

In dewatering terminology, wellpoints (Chapter 9), deep wells (Chapter 10), and ejectors (Section 11.2) are all types of wells, categorized by their method of pumping. A "sump" (Chapter 8) is not considered a well because it is effectively a more or less crude pit to allow the collection of water.

3.5.1 Radial flow to wells

One of the defining features of flow through an aquifer toward a well is that flow converges radially as it passes through an ever smaller cross-sectional area of aquifer, resulting in a corresponding increase in flow velocity as the well is approached (Figure 7.4a). A drawdown curve is generated around the well as a result of pumping; in three dimensions around the well, the drawdown curve describes a conical shape known as the "cone of depression." The limit of the cone of depression defines the "zone of influence" of a well.

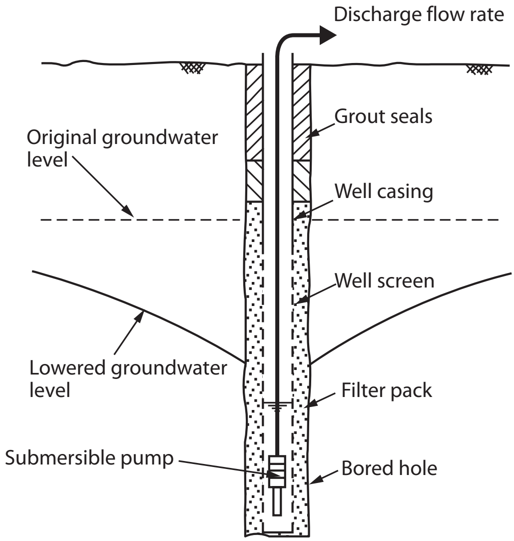


Figure 3.12 Principal components of a groundwater lowering well.

The effect of converging flow is compounded in unconfined aquifers because the drawdown at the well effectively reduces the aquifer thickness at the well (Figure 3.7), further reducing the area available for groundwater flow. This is one of the reasons why flow rate equations for unconfined aquifers are more complex than for confined aquifers.

Convergence of flow, and the associated high groundwater velocities, leads to the phenomenon of "well loss," in which the water level inside a pumped well may be significantly lower than in the aquifer immediately outside. In some circumstances, large well losses can be a sign of poorly designed or poorly constructed wells.

3.5.2 Zone of influence

The zone of influence is a theoretical concept used to visualize how a well is affecting the surrounding aquifer. Imagine a well penetrating an aquifer that has an initial water table or piezometric level at the same elevation everywhere (Figure 3.13). When water is first pumped from the well, the water level in the well will be lowered, and flow will occur from the aquifer into the well. This water will be water released from storage in the aquifer around the well. As time passes, the cone of depression will expand away from the well, releasing additional water from storage. The zone of influence will continue to increase with time, but at a diminishing rate until either an aquifer boundary is reached or the infiltration recharge into the aquifer within the zone of influence is sufficient to supply the yield from the well.

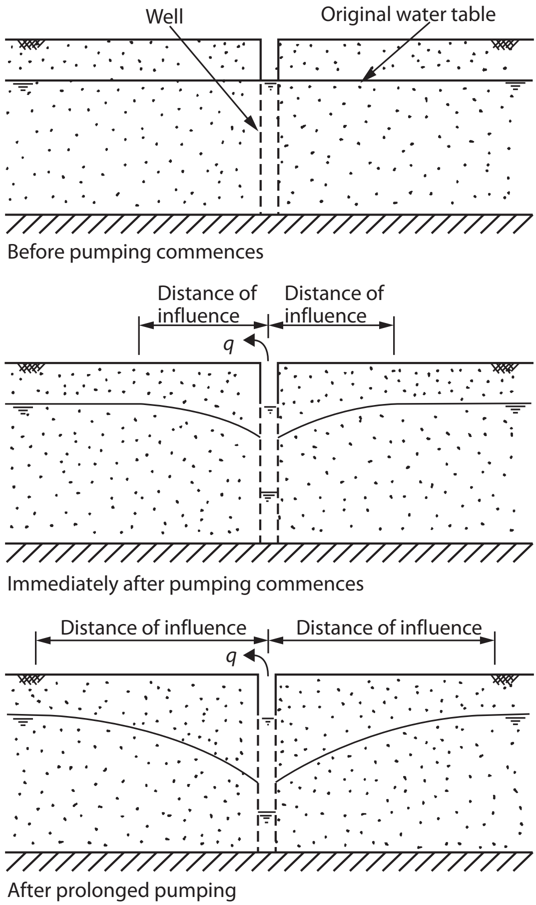


Figure 3.13 Zone of influence.

An idealized zone of influence is perfectly circular. The size of the zone is described by the "distance of influence," which is the distance from the center of the well to the outside edge of the zone where drawdown occurs.

Even if no source of recharge is encountered by the expanding zone of influence, after some time of pumping, it will be expanding so slowly that it is effectively constant and is said to have reached a quasi-steady state. This is the approach assumed in several of the design methods described in Chapter 7. This simplifies all the sources of water (whether from storage or external recharge) as being equivalent to a single circular source of water at distance R_0 from the well. R_0 is the distance of influence for radial flow, sometimes known as the "radius of influence." The distance of influence for plane flow (e.g., for flow to line of wellpoints) is given the symbol L_0.

The distance of influence is an important parameter when estimating the discharge flow rate from a well or dewatering system. All other things being equal, a small distance of influence will predict a higher flow rate than a large distance of influence. It is vital that realistic values of distance of influence are used in calculations—poor selection of this parameter is one of the prime causes of errors in flow rate calculations. The important points to consider include:

1. The steady state distance of influence used in many calculations is a theoretical concept only. The true distance of influence is zero when pumping begins and increases with time.
2. The distance of influence will generally be greater in high-permeability soils than in low-permeability soils.
3. The distance of influence will generally be greater for larger well drawdowns than for small drawdowns.

It may be possible to determine the true distance of influence from appropriately analyzed well pumping tests (see Section 6.7). There are, however, a number of equations available to estimate R_0 and L_0. Two of the most commonly used are given here.

The empirical formula developed by Weber, but commonly known as Sichardt's formula, allows R_0 to be estimated from the drawdown s and aquifer permeability k.

$$R_0 = Cs\sqrt{k} \tag{3.4}$$

where C is an empirical factor. For radial flow to a well, if s is in meters and k is in meters per second, R_0 can be obtained in meters using a C value of 3000. The Sichardt estimate of R_0 is a quasi-steady state value and does not take into account the time period of pumping. The time-dependent development of R_0 is described by the formula of Cooper and Jacob (1946).

$$R_0 = \sqrt{\frac{2.25kDt}{S}} \tag{3.5}$$

where D is the aquifer thickness, k and S are the aquifer permeability and storage coefficients, respectively, and t is the time since pumping began.

When using distance of influence in design calculations, it is important to be vigilant for unrealistic values of R_0 and L_0, especially very small or very large values. In the authors' experience, values approximately less than 30 m or more than 5000 m are rare and should be viewed with caution. It may be appropriate to carry out sensitivity analyses using a range of distance of influence values to see the effect on calculated flow rates.

Another problem can occur if the system is designed using the long-term R_0 or L_0. This will predict a much lower flow rate than may be generated during the initial period of pumping when R_0 is small as the cone of depression is expanding. It may be appropriate to design for a smaller, short-term R_0, or to design using the long-term R_0, but provide spare pumping capacity, over and above that predicted, to deal with the higher flows during the initial drawdown period.

3.5.3 Well losses

In general, the water level inside the screen or casing of a pumped well will be lower than in the aquifer immediately outside the well. This difference in level is known as the "well loss" and results from the energy lost as the flow of groundwater converges toward the well. The drawdown observed in a pumped well has two components (Figure 3.14).

1. The drawdown in the aquifer. This is the drawdown resulting from laminar (smooth) flow in the aquifer, and is sometimes known as the aquifer loss. This component is normally assumed to be proportional to flow rate q.
2. The well loss. This is the drawdown resulting from resistance to turbulent flow in the aquifer immediately outside the well, and through the well screen and filter pack. This component is normally assumed to be proportional to q^n, and so increases rapidly as the flow velocity increases.

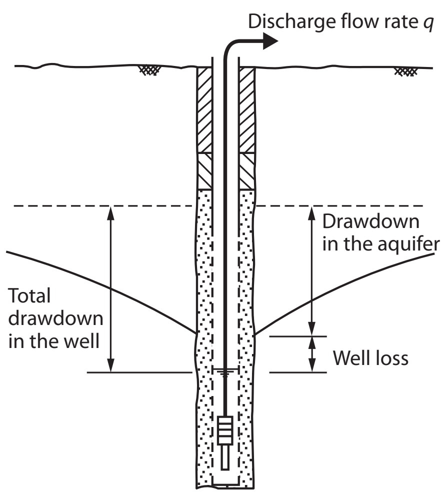


Figure 3.14 Well losses.

Well loss means that the water levels observed in a pumped well are unlikely to be representative of the aquifer in the vicinity of the well. Drawdowns estimated from pumped wells will tend to overestimate drawdowns. It is preferable to have dedicated monitoring or observation wells, or at least to observe drawdown in some of the wells that are not being pumped.

Jacob (1946) suggested that in many cases, $n = 2$. This is now widely accepted and allows the drawdown s_w in a well to be expressed as

$$s_w = Bq + Cq^2 \tag{3.6}$$

where B and C are calibration coefficients (which can be determined from the results of step drawdown tests; Clark 1977). The Bq term is the aquifer drawdown, and Cq^2 is the well loss.

If a well experiences high well loss, its yield will be limited. At high flow rates, the drawdown inside the well will be large, but drawdown in the aquifer (which is the aim of any groundwater lowering system) may be much smaller. In poorly performing wells, it is not unusual for the drawdown outside a well to be less than half that inside the well. Well losses can be minimized by designing the well with sufficient “wetted screen length” and with a low screen entrance velocity (see Section 10.3). Ensuring that the well is adequately developed (see Section 10.7) can help reduce well losses.

3.5.4 Effect of diameter of well

Common sense suggests that, other things being equal, a larger diameter well will have the ability to yield more water than a smaller diameter well. This is because the well will have more contact area with the aquifer, and also because the flowing water has to converge less to reach a larger well.

However, the relationship between yield and diameter is not straightforward. It is worth stating that doubling the diameter of the well will not double the potential yield from the well, and may only increase the yield by a small proportion. Table 3.3, based on work by Ineson (1959), shows the estimated relation between yield and bored diameter (the drilled diameter through the aquifer, not the diameter of the well screen). This indicates, for example, that in a homogeneous aquifer, increasing the well diameter by 50% from approximately 200 mm to approximately 300 mm will increase the yield by only 11%. The increase in yield is more marked in fissured aquifers because a larger diameter well has an increased chance of intercepting water-bearing fissures. The true relationship may be more complex than this, especially for wells with high losses. In those cases, the increase in diameter, which will reduce groundwater flow velocity, may help reduce well losses and attainable well yield may increase rather more than that shown in Table 3.3.

Table 3.3 Effect of well diameter on yield

Bored diameter of well through aquifer	*203 mm*	*305 mm*	*406 mm*	*457 mm*	*610 mm*
Homogeneous aquifer (intergranular flow)	1.00	1.11	1.21	1.23	1.32
Fissured aquifer	1.00	1.29	1.52	1.61	1.84

Source: Ineson, J., *Proceedings of the Institution of Civil Engineers*, 13, July, 299–316, 1959. With permission.

In practice, the economics of drilling and lining wells means that temporary works groundwater lowering wells are rarely constructed at diameters of greater than 300–450 mm. Well diameters are often chosen primarily to ensure that the proposed size of electrosubmersible pump can be installed in the well (see Section 10.3).

3.5.5 Equivalent wells and lines of wells

The foregoing discussions have concentrated on a single pumped well in isolation. In reality, most groundwater lowering is carried out using several closely spaced pumped wells (be they wellpoints, deep wells, or ejectors) acting in concert. With this approach, the cone of depression of each well overlaps (or interferes), creating additional drawdown. This effect is known as the superposition of drawdown and can create groundwater lowering over wide areas, which is ideal to allow excavations to be made below the original groundwater levels (Figure 3.15).

Wells are normally installed either as rings around an excavation or as lines alongside one or more sides of the dig. It is probably theoretically possible to consider the influence of each well individually and determine the complex interaction between the wells, but this would be tedious, especially when a large number of wells are involved. A more practical, and

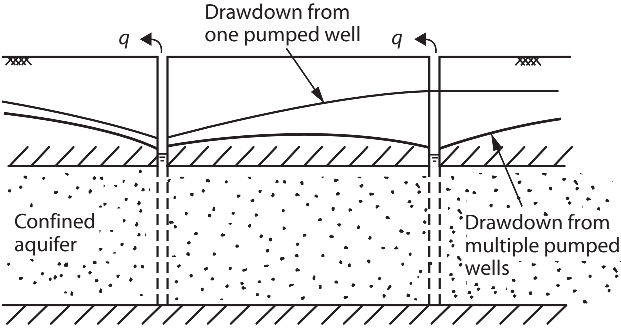


Figure 3.15 Superposition of drawdown.

empirically very effective, approach is to consider a dewatering system of many wells to act on a gross scale as a large equivalent well or slot.

Circular or rectangular rings of wells can be thought of as large equivalent wells (see Figure 7.5) to which flow is, on a gross scale, radial. Similarly, lines of closely spaced wells (such as wellpoints alongside a trench) can be thought of as equivalent slots (Figure 7.6) to which flow is predominantly plane to the sides. Once this conceptual leap is made, the well and slot formula given in Chapter 7 can be used to model the overall behavior of dewatering systems, without having to consider each well individually. Formulae for estimating the size of an equivalent well from the dimensions of a dewatering system are given in Section 7.6.

3.6 AQUIFERS AND GEOLOGICAL STRUCTURE

The aquifer types described in the foregoing sections are a theoretical ideal. At many sites, more than one aquifer may be present (perhaps separated by aquicludes or aquitards), or the aquifers may be of finite extent and be influenced by their boundaries. The particular construction problems resulting from the presence of aquifers at a site will be strongly influenced by the soil stratification and geological structure.

This section will illustrate the importance of an appreciation of geological structures to the execution of groundwater lowering works. Two case histories will be presented: the London Basin shows how large-scale geological structures can allow multiple aquifer systems to exist; and a problem of base heave in a small trench excavation illustrates the importance of smaller-scale geological details.

3.6.1 Multiple aquifers beneath London

The city of London is founded on river gravels and alluvial deposits associated with the River Thames, which are underlain by very low-permeability London clay. These gravels form a shallow (generally <10 m thick) aquifer. Construction of utility pipelines, basements, and other shallow structures often requires groundwater lowering to be used; wellpointing and deep wells have proved to be effective expedients in these conditions. However, the geology beneath London allows another, deeper, aquifer to exist below the city, largely isolated from the shallow aquifer.

Beneath the London clay lie a series of sands and clays comprising the Lambeth Group stratum (formerly known as the Woolwich and Reading Beds) and the Thanet Sand stratum. These are underlain by the Chalk, a fissured white or grey limestone, which rests on the very low-permeability Gault Clay. The overall geological structure is a syncline forming what is often called the “London Basin.” The Chalk, Thanet Sand, and parts of the Lambeth Group together form an aquifer. The upper 60–100 m of the Chalk are probably the dominant part of the aquifer, where significant fissure networks readily yield water to wells. The sands are of moderate permeability and generally do not yield as much water as the Chalk. The overlying London clay acts as an aquiclude or confining bed, effectively separating the deep aquifer from the shallow gravel aquifer.

The Chalk has a wide exposure on the North Downs to the South of London and on the Chilterns to the North, and occurs as a continuous layer beneath the Thames Valley (Figure 3.16). Rain falling on central London may ultimately reach the gravel aquifer, but the London clay prevents it from percolating down to the Chalk. The Chalk obtains its recharge from rain falling on the North Downs and the Chilterns many miles from the city. Ultimately, this water forms part of the reservoir of water in the chalk aquifer. If the recharge exceeds the discharge from the aquifer (from wells and natural discharge to springs and the River Thames) the water pressure in the aquifer will rise slowly. If discharges exceed the recharge the water pressure will fall.

Before London developed as a city, the natural rates of recharge and discharge meant that the deep aquifer had sufficient water pressure for it to act as a confined aquifer (see Section 3.4). In the lower lying areas of the city, there was originally sufficient pressure in the aquifer to allow a well drilled through the London Clay into the Chalk to overflow naturally as a flowing artesian well. In fact, in central London, there are still a few public houses called the *Artesian Well*, indicating that in earlier days, the locals were probably supplied with water from a flowing well.

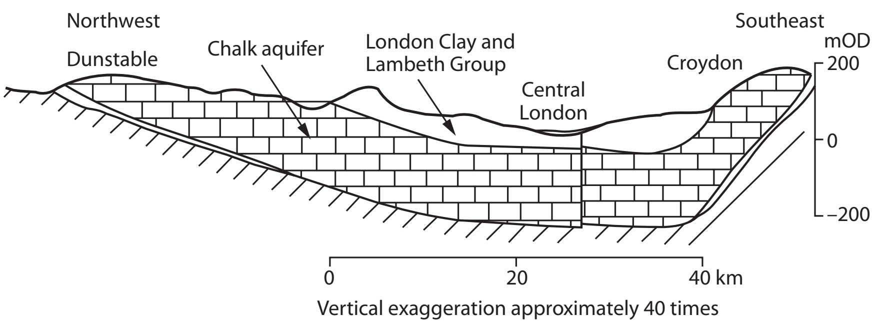


Figure 3.16 Chalk aquifer beneath London. The chalk aquifer extends beneath the London basin and receives recharge from the unconfined areas to the north and south. The London clay deposits act as a confining layer beneath central London. Before the twentieth century, flowing artesian conditions existed in many parts of the city. (After Sumbler, M.G. *British Regional Geology: London and the Thames Valley*, 4th ed. HMSO, London, 1996.)

This availability of groundwater led to a large number of wells being drilled into the deep aquifer (where the water quality was more "wholesome" than in the gravel aquifer). Rates of groundwater pumping increased during the eighteenth, nineteenth and early part of the twentieth centuries. This resulted in a significant decline in the piezometric level of the deep aquifer. Artesian wells ceased to flow, pumps had to be installed to allow water to continue to be obtained and, over the years, the pumps had to be installed lower and lower to avoid running dry. By the 1960s, the water levels in wells in some areas of London were 90 m below the ground surface—a huge drop relative to the original artesian conditions. In some locations, the water pressure was reduced below the base of the London Clay, so the formerly confined aquifer became unconfined. The deeper water levels increased pumping costs and made well supplies less cost-effective compared with mains water. This, together with a general relocation of large water-using industries away from central London, has resulted in a significant reduction in groundwater abstraction. As a result, the piezometric level in the aquifer has recovered since then (at more than 1 m/yr at some locations during the 1980s; see Figure 17.1). By the 1990s, the piezometric level in many areas was within 55 m of ground level.

This continuing rise of water pressure is a major concern because much of the deep infrastructure beneath London (deep basements, railway, and utility tunnels) were built during the first half of the twentieth century, when water pressures were at an all-time low. Several studies (e.g., Simpson et al. 1989) have addressed the risk of flooding or overstressing of existing deep structures if water levels continue to rise. The management of water levels beneath London (and indeed, beneath other major cities around the world) is an important challenge to be faced by groundwater specialists during the first half of the twenty-first century. The use of permanent dewatering systems as part of the strategy to deal with rising groundwater levels is discussed in Chapter 14.

In a construction context, an appreciation of the aquifer system is vital to ensure that deep structures are provided with suitable temporary works dewatering. Figure 3.17 shows a typical arrangement as might be used for a deep shaft structure in central London. Important points to note are:

1. The structure penetrates two aquifers, separated by an aquiclude. Groundwater will need to be dealt with separately in each aquifer.
2. In London, it is common to deal with the upper aquifer by constructing a cutoff wall (see Chapter 12), penetrating to the London Clay, excluding shallow groundwater. This is possible because London clay is at a relatively shallow depth. If clay was present only at greater depth, any cutoff would need to be deeper and it may be more economic to dewater the upper aquifer.

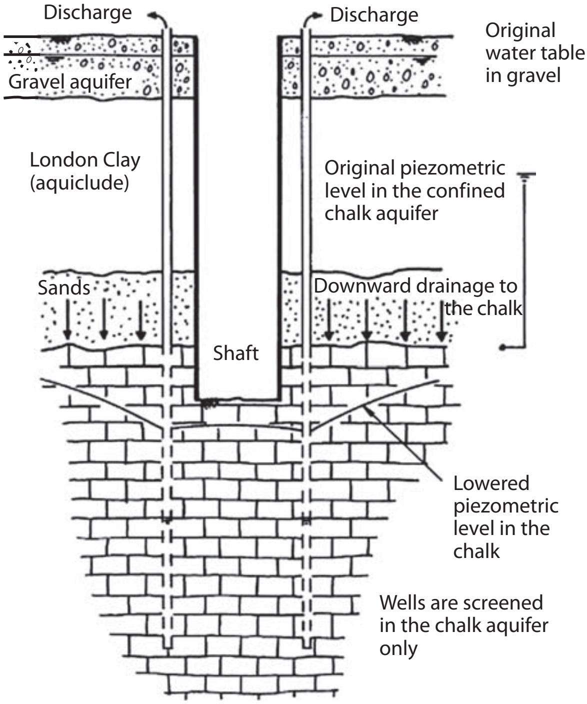


Figure 3.17 Groundwater control in multiple aquifers.

3. Wells are used to pump from the deep aquifer to lower the piezometric level to a suitable distance below the excavation. Because the wells must be relatively deep (perhaps up to 100 m), and therefore, costly, it is important to design the wells and pumps to have the maximum yield possible, so that the number of wells can be minimized.
4. Although the lower aquifer consists of both the Chalk and the various sand layers between the top of the Chalk and the base of the London Clay, wells are often designed to be screened in the Chalk only, and are sealed from the sands using casing. This is because it can be difficult to construct effective well filters in the sands (which are fine-grained and variable), yet wells screened in the Chalk are simpler to construct and can be very efficient, especially if developed by acidization (see Section 10.7). This is the approach successfully adopted for some structures on the London Underground Jubilee Line Extension project described by Linney and Withers (1998). Excavations in the Thanet Sand were dewatered without pumping directly from the sand, but by pumping purely from the underlying

Chalk. This might seem a rather contradictory approach, but is an example of the "underdrainage" method. This is a way of using the geological structure to advantage by pumping from a more permeable layer beneath the layer that needs to be dewatered; the upper poorly draining layer will drain down into the more permeable layer (see Section 7.6).

3.6.2 Water pressures trapped beneath a trench excavation

Dr. W. H. Ward (1957) reported some construction difficulties encountered by a contractor excavating a pipeline trench near Southampton. The trench excavation was made through an unconfined aquifer of sandy gravel overlying the clays of the Bracklesham Beds. The contractor dealt with the water in the sandy gravel by using steel sheet piling to form a cutoff on either side of the trench and exclude the groundwater. The clay in the base of the excavation was not yielding water, so dewatering measures were not adopted.

The trench was approximately 6.1 m deep, to allow placement of a 760 mm diameter pipe which was laid on a 150-mm-thick concrete slab in the base of the excavation. The construction difficulties encountered consisted of uplift of the bottom of the trench (called "base heave," see Section 4.6), often occurring overnight while the trench was open. At one location, the trench formation rose by almost 150 mm before the concrete slab was cast, and a further 50 mm after casting.

When Dr. Ward and his colleagues at the Building Research Station were consulted, they suggested that the problem might be due to a high groundwater pressure in a water-bearing stratum below the base of the trench. This was proved to be the case when a small borehole was drilled in the base of the trench. This borehole overflowed into the trench with the flowing water bringing fine sand with it. The water pressure in the borehole was later determined to be at least 1.3 m above the trench formation level, but it is likely that the original piezometric level was even higher, because the flowing discharge from the borehole may have reduced pressures somewhat. Once the problem had been identified, the contractor was able to complete the works satisfactorily by installing a system of gravel-filled relief wells (see Section 11.5) in the base of the trench to bleed off the excess groundwater pressures (Figure 3.18).

This case history illustrates the importance of identifying the small-scale geological structure around an excavation. It seems that in this case the trench was excavated through the upper aquifer (the sandy gravel), which was dealt with using a sheet pile cutoff, and the base of the trench was dug

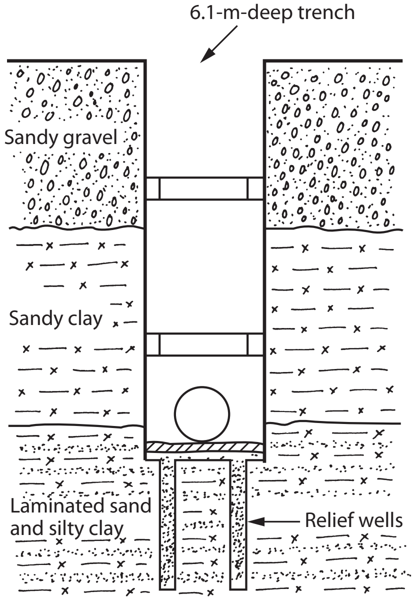


Figure 3.18 Use of simple relief wells to maintain base stability. (After Ward, W.H. *Géotechnique*, 7, 3, 134–139, 1957.)

into low-permeability clay, which is effectively an aquiclude. The problems occurred because there was a separate confined aquifer beneath the aquiclude, which contained sufficient water pressure to lift the trench formation. Once identified and understood, the problem was solved easily using relief wells. However, because the problem was not identified in the site investigation before work started, time and money was wasted in changing the temporary works, as well as repairing the damaged pipelines. A classic failing of site investigations for groundwater lowering projects is that the boreholes are not taken deep enough to identify any confined aquifers which may exist beneath the proposed excavations. Guidelines on suitable depths for boreholes are given in Section 6.5.

3.7 AQUIFER BOUNDARIES

Civil and geotechnical engineers can readily appreciate the importance of permeability in the design of groundwater lowering systems—permeability is a traditional numerical engineering value. However, engineers are sometimes less proficient in recognizing the less quantifiable effects that aquifer boundary conditions have on groundwater flow to excavations. The following sections will outline some important aquifer boundary conditions.

3.7.1 Interaction between aquifers and surface water

It is obvious from the hydrological cycle (see Section 3.2) that groundwater is inextricably linked with surface waters such as rivers, streams, and lakes. The significance of the link between groundwater and surface water at a given site depends on the geological and hydrological setting.

Where surface water flows across or sits on top of an aquifer, water will flow from one to the other—the direction of flow will depend on the relative hydraulic head. The magnitude of the flow will be controlled by Darcy's law, and will be affected by the permeability and thickness of any bed sediment, and the head difference between the aquifer and the surface water. Bodies of surface water that are quiet or slow-flowing may have low-permeability bed sediments, which may dramatically reduce flow between surface and groundwater. Similarly, if the surface water is sitting on an aquitard or aquiclude, it may be effectively isolated from groundwater in underlying aquifers.

Where watercourses (such as rivers and streams) are connected to aquifers they commonly receive water from the aquifer (the water entering the river from groundwater is termed "baseflow"). Such a river is said to be a "gaining" river (Figure 3.19a). This is perhaps contrary to many people's

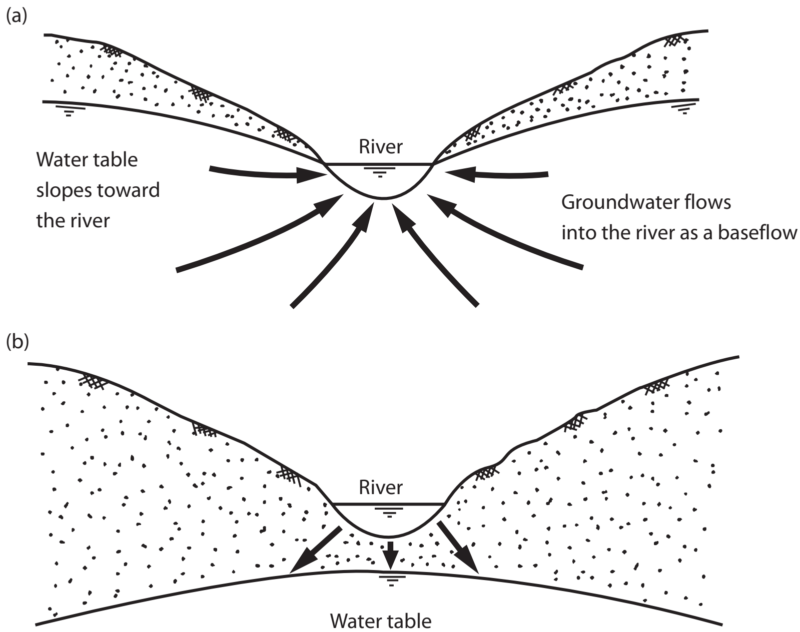


Figure 3.19 Interaction between rivers and aquifers. (a) Cross section through a gaining river. (b) Cross section through a losing river.

expectations, that rivers should feed groundwater, rather than vice versa. "Losing" rivers do sometimes exist, especially in unconfined aquifers with deep water tables (Figure 3.19b). If groundwater lowering is carried out near a "gaining" river, the hydraulic gradients may be reversed, changing a normally gaining river to a losing one. Even if the hydraulic gradients do not reverse, groundwater lowering may reduce baseflow to a gaining river. If pumping continues for an extended period, this may reduce river flow, and perhaps result in environmental concerns (see Section 15.4).

Lakes often have low-permeability silt beds, reducing the link with groundwater, although wave action near the shore may remove sediment, allowing increased flow into or out of the groundwater body. Man-made lagoons or dock structures may have silt beds (especially if they are relatively old) or may have linings or walls of some sort; however, just because linings exist, it does not mean that they do not leak! Analysis of a pumping test (Section 6.7) can be an effective way of determining whether groundwater lowering will be significantly influenced by any local bodies of surface water.

3.7.2 Interaction between aquifers

In the same way that there can be flow between groundwater and surface water, groundwater can flow between aquifers if hydraulic head differences exist between them.

Many aquifer systems exist under "hydrostatic conditions"—this is the case when the hydraulic head is constant with depth, so that observation wells installed at different depths show the same level (Figure 3.20a). In this case, there would be no vertical flow of groundwater between shallow and deep aquifers separated by an aquitard. But sometimes non-hydrostatic conditions exist. In Figure 3.20b, the hydraulic head in the deep aquifer is greater than the shallow aquifer, so water will flow upward from the deep to the shallow aquifer. The flow may be very slow if the aquitard is of low permeability; if the stratum between the aquifers is an aquiclude, the flow will be so small that it is often ignored in the analysis of short-term groundwater lowering installations.

Inter-aquifer flow may be a long-term phenomenon, perhaps sustained by different recharge sources for each aquifer. On the other hand, it may be a short-term condition resulting from temporary groundwater lowering operations disturbing the groundwater regime (this artificially induced flow between aquifers is one possible environmental effect of dewatering, see Section 15.4).

3.7.3 Recharge boundaries

Zones or features where water can flow into an aquifer are termed "recharge boundaries," some commonly occurring examples of which are shown in

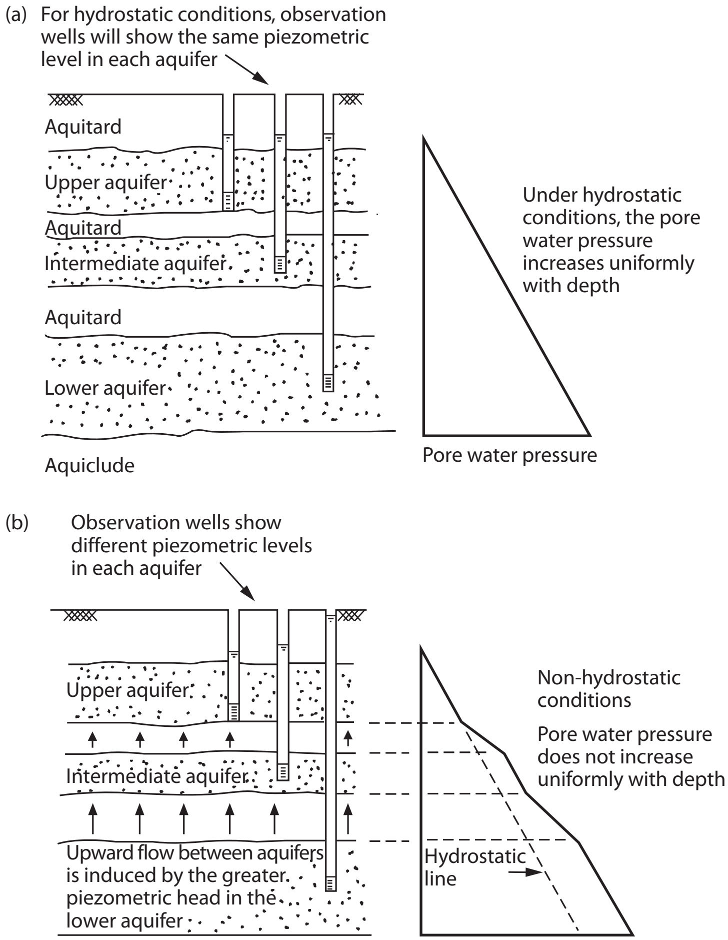


Figure 3.20 Interaction between aquifers. (a) Hydrostatic conditions. (b) Flow between aquifers.

Figure 3.21. If recharge boundaries exist within the distance of influence they can have a significant effect on the behavior of dewatering schemes. They may cause the cone of depression to become asymmetric (because the extent of the cone will be curtailed where it meets the recharge source). The flow rate that must be pumped by a dewatering system will often be increased by the presence of a recharge boundary. It is essential that any potential recharge boundaries be considered during the investigation and design of dewatering works.

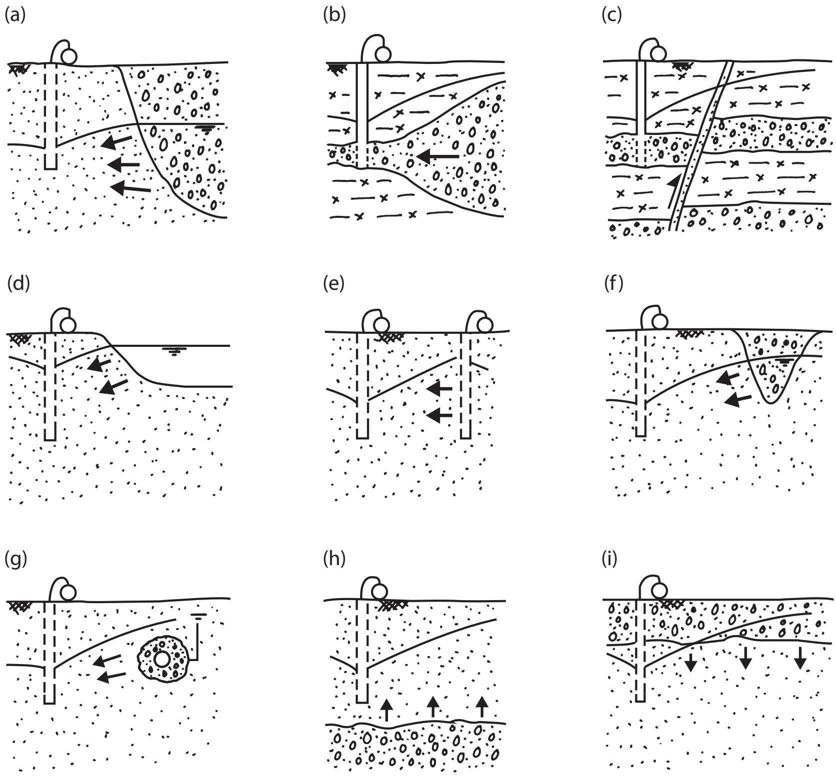


Figure 3.21 Potential aquifer recharge boundaries. (a) High-permeability zone. (b) Increase in aquifer thickness. (c) Fault. (d) Surface water in hydraulic connection with the aquifer. (e) Recharge wells. (f) Gravel lens or channel. (g) Flooded land drain or sewer bedding. (h) Underlying high-permeability stratum. (i) Overlying high-permeability stratum.

3.7.4 Barrier boundaries

Real aquifers are rarely of infinite extent and may be bounded by features which form barriers to groundwater flow. Figure 3.22 shows some commonly occurring barrier boundaries. The presence of barrier boundaries will tend to reduce the pumped flow rate necessary to achieve the required drawdown.

3.7.5 Discharge boundaries

Water can sometimes be discharged naturally from aquifers. Water will flow from an unconfined aquifer if the water table intersects the ground

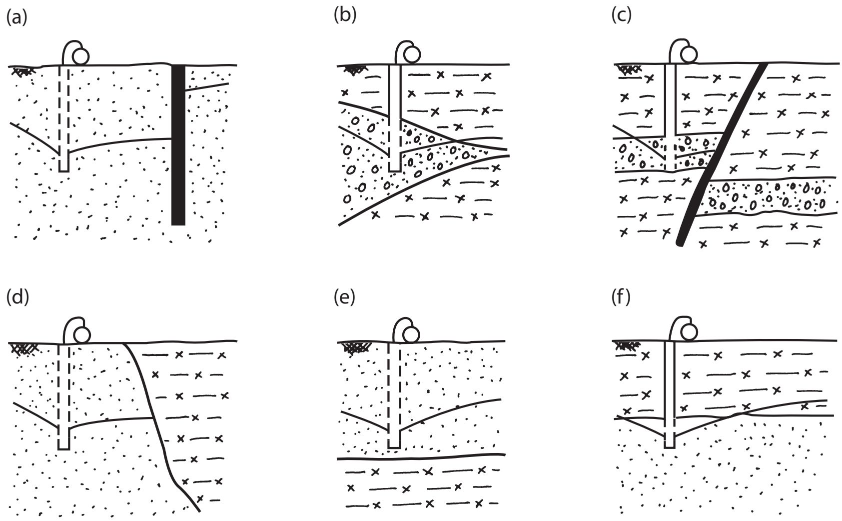


Figure 3.22 Potential aquifer barrier boundaries. (a) Partial cutoff wall. (b) Reduction in aquifer thickness. (c) Fault. (d) Low-permeability zone. (e) Underlying low-permeability stratum. (f) Overlying low-permeability stratum.

surface. Diffuse discharges are called seepages or, if the flow is very localized (perhaps at a fault or fissure), the discharge is termed a spring. Flowing artesian aquifers can also discharge if faults or fissures allow water a path to the surface. Water flowing between aquifers may also constitute a discharge boundary condition.

Man's influence, in the form of pumping from wells (either for supply or for groundwater lowering for construction, quarrying, mining, etc.) can also create discharge boundaries. Figure 3.23 shows some examples of discharge boundaries.

3.8 USING GEOLOGICAL STRUCTURE TO ADVANTAGE

Previous sections have described how the boundaries and structure of aquifers and aquitards can complicate the flow of groundwater. The presence of recharge boundaries or multiple layered aquifers will have a significant effect on the groundwater lowering requirements for an excavation. It is essential that, as the first step in any design process, a "conceptual model" of groundwater conditions is developed. The background to conceptual models is described in Section 7.4.

Although geological structures and boundaries can make dewatering more difficult, conversely, a designer can try to use these features to

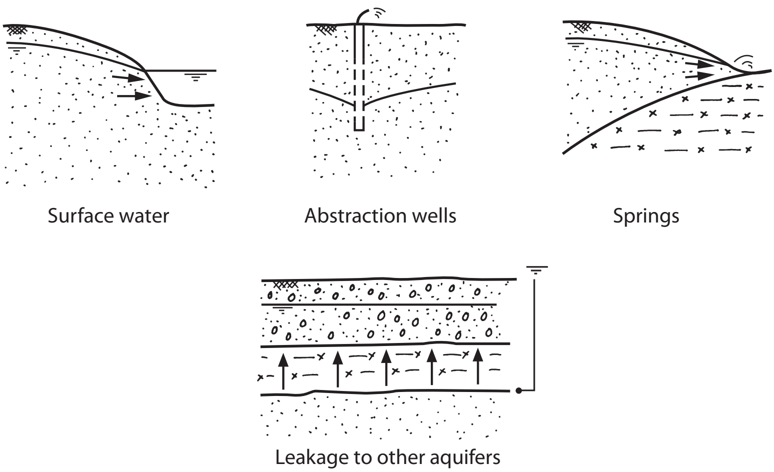


Figure 3.23 Potential aquifer discharge boundaries.

advantage. For example, if the conceptual model highlights the presence of a more permeable layer within the aquifer sequence, by far the most effective approach is to abstract water directly from the permeable layer. This will maximize well yields and will induce the adjacent, less permeable, layers to drain to the permeable layer. Where the permeable layer is at depth, this approach of pumping preferentially from the most permeable layer is known as underdrainage (see Section 7.6 and Figure 7.9).

3.9 GROUNDWATER CHEMISTRY

The study of the chemical composition of groundwater is a key area of hydrogeology, helping to ensure that water abstracted for potable use is safe to drink or fit for its intended use. The water abstracted for groundwater lowering purposes is rarely put to any use, and, subject to the necessary permissions (see Chapter 17), is typically discharged to waste. Accordingly, water chemistry has often been thought of as an irrelevance to the dewatering practitioner.

However, a basic knowledge of groundwater chemistry can be useful for the following reasons:

1. Discharge consents and permissions. In many locations, permission to dispose of the discharge water to a sewer or surface watercourse will only be granted if it can be demonstrated that the water is of adequate quality.
2. Corrosion and encrustation. Water chemistry will influence well clogging, encrustation, corrosion, and biofouling (see Section 16.9) that

may affect systems in long-term operation. Clogging and encrustation can be a particular problem for artificial recharge systems (see Section 11.9).

3. The effect of groundwater lowering on water quality. Abstracting water from an aquifer may affect existing groundwater quality. Monitoring may be necessary to determine if these effects are occurring, and to control any mitigation measures.

If a project is likely to require detailed study of water chemistry, specialist advice should be obtained. This section is intended only to highlight some of the important practical issues of water chemistry relevant to groundwater lowering. Readers interested in the field are commended to Lloyd and Heathcote (1985) as an introductory text.

3.9.1 Chemical composition of groundwater

Water is a powerful solvent, and a wide range of substances will dissolve in it to some degree. Almost all water in the hydrological cycle will contain some dissolved minerals or other substances. Even rainwater contains dissolved carbon dioxide and some sodium chloride lifted into the atmosphere from the oceans.

Substances dissolved in water exist as electrically charged atoms or molecules known as ions. Positively charged ions are known as anions and negatively charged ions are known as cations. For example, when sodium chloride (common salt, NaCl, which is highly soluble) dissolves in water, it exists as sodium anions (Na^+) and chloride cations (Cl^-).

There are certain substances which commonly exist at substantial concentrations (several milligrams per liter, mg/L) in natural groundwater. These are known as the major anions and cations (Table 3.4), which are routinely tested for in groundwater analyses. A variety of trace metals are also present (called trace because they are generally present in much

Table 3.4 Major anions and cations in groundwater

Major anions	*Major cations*
Sodium (Na^+)	Bicarbonate (HCO_3^-)
Potassium (K^+)	Carbonate (CO_3^-)
Calcium (Ca^{2+})	Sulfate (SO_4^{2-})
Magnesium (Mg^{2+})	Chloride (Cl^-)
	Nitrate (NO_3^-)
	Nitrite (NO_2^-)
	Ammonia (NH_3)
	Phosphate (PO_2^-)

smaller concentrations, perhaps only a fraction of a milligram per liter). For groundwater lowering purposes, the most important trace metals are iron and manganese because they influence the severity of encrustation and biofouling (see Section 16.9).

In addition to dissolved natural minerals and compounds, if groundwater contamination has occurred, other (potentially harmful) substances may be present. Many of the most problematic substances are said to be "organic compounds"; this does not mean that they result from natural growth, but means that they are based around molecules formed from carbon atoms, the building blocks of organic life. Some of the organic compounds (which include pesticides) are toxic even at extremely low levels (<1 μg/L). At these low concentrations, detection of these compounds requires the highest quality of sampling and testing; specialist advice is essential. Further details on contamination problems can be found in Fetter (1993).

It is apparent that almost no groundwater is "pure," if by pure we mean containing nothing but H_2O! Nevertheless, water that contains relatively little dissolved material is said to be "fresh." Most groundwater abstracted for drinking use is classified as fresh, and requires little treatment other than basic sterilization to kill any harmful bacteria present. Even water suitable for industrial use (such as steam raising in a boiler) tends to be relatively fresh because the lower mineral content reduces the buildup of scale deposits within the pipework. In many developed countries, permissible limits are set for the chemical composition of water that can be used for human consumption; guidelines are also available for water to be used for cultivation of crops or livestock (see Lloyd and Heathcote 1985, Chapter 10; Brassington 1995, Chapter 6).

3.9.2 Field monitoring of groundwater chemistry

In principle, the most obvious way to investigate groundwater chemistry is to obtain a sample of groundwater, seal it into a bottle, and send it to a suitably equipped laboratory for testing. It might be appropriate to test for major anions and cations, selected trace metals, and any other substances of interest at the site in question.

For groundwater lowering systems, obtaining a sample is relatively straightforward. It may be possible to fill a sample bottle directly at the discharge tank or via sample tap at the wellhead. However, when taking groundwater samples from the discharge flow, the following factors should be considered:

1. Try to minimize the exposure of the sample to the atmosphere. Try and obtain it directly from the discharge of the pump or dewatering system. Totally fill the bottle and try and avoid leaving any air inside when it is sealed. If the pump discharge is "cascading" before the

sampling point, the water will become aerated and oxidation may occur. The discharge arrangements should be altered so the sample can be obtained before aeration occurs.
2. Samples may degrade between sampling and testing. The samples should be tested as soon as possible after they are taken and, ideally, should be refrigerated in the meantime. The bottles used for sampling should be clean with a good seal. However, the sample may degrade while in the bottle (for example, by trace metals oxidizing and precipitating out of solution). Specialists may be able to provide advice on the addition of suitable preservatives to prevent this from occurring. The choice of sample bottle (glass or plastic) should also be discussed with the laboratory because some test results can be influenced by the material of the sample bottle.
3. Use an accredited, experienced laboratory.

Sometimes groundwater samples may be required from a site when there is no dewatering pumping taking place—perhaps for pre- or postconstruction background monitoring. In that case, a sampling pump will have to be used to obtain a sample from an observation well. The water standing in the well has been exposed to the atmosphere and is unlikely to represent the true aquifer water chemistry. Therefore, it is vital to fully “purge” the well before taking a sample. Purging involves pumping the observation well at a steady rate until at least three “well volumes” of water have been removed (a well volume is the volume of water originally contained inside the well liner). Specialist sampling pumps should be used in preference to airlifting because the latter method may aerate the sample, increasing the risk of oxidation of trace metals and other substances.

If samples are taken to an off-site laboratory, it may be several days before the results are ready. Even if the tests are rushed through as priority work, some of the actual procedures may take a week or more. Comprehensive testing of samples is not cheap, either; a reasonably complete suite of testing may cost several hundred pounds per sample at 2012 prices. A good way of reducing costs, and obtaining rapid results, is to take and test water samples on a periodic basis (perhaps monthly), but carry out daily (or, using a datalogger, continuously) monitoring of the “wellhead chemistry.”

Wellhead chemistry is a hydrogeological term used to describe certain parameters, which are best measured as soon as the water is pumped from the well; for example, at the wellhead. It is best to measure these parameters here as they are likely to change during sampling and storage, which makes laboratory-determined values less representative. Typical wellhead chemistry parameters include:

- Specific conductivity, EC
- Water temperature

- pH
- Redox potential, E_H
- Dissolved oxygen, DO

Perhaps the most commonly measured wellhead parameter for groundwater lowering systems is specific conductivity, EC. Specific conductivity is a measure of the ability of the water to conduct electricity and is a function of the concentration and charge of the dissolved ions; it is reported in units of microsiemens per centimeter (μS/cm) corrected to a reference temperature of 25°C. EC is useful in that it can be related to the amount of total dissolved solids (TDS) of the water (Lloyd and Heathcote 1985).

$$\mathrm{TDS} = k_e \mathrm{EC} \tag{3.7}$$

where:

TDS = total dissolved solids (in milligrams per liter)
EC = specific conductivity (in microsiemens per centimeter) at 25°C
k_e = a calibration factor with values between 0.55 and 0.80 depending on the ionic composition of the water

TDS and EC have been related to each other for various water classifications in Table 3.5. Fresh water will have a low TDS (most water supply boreholes for potable use produce water with a TDS of no more than a few hundred milligrams per liter). The higher the TDS, the less fresh the water. The term "saline" is used for convention's sake and does not necessarily imply that a high TDS is the result of saline intrusion (see Section 15.4). A high TDS may be an indicator of highly mineralized waters that have been resident in the ground for very long periods, slowly leaching minerals from the soils and rocks.

Daily monitoring of EC using a conductivity probe has proved useful on sites where there was concern that water of poorer quality (e.g., from saline intrusion or from deeper parts of the aquifer) might be drawn toward the pumping wells. Readings of conductivity were taken every day, and reviewed for any sudden or gradual changes in EC, which would have indicated that poor quality water was reaching the wells. In effect, the EC monitoring was used as an early warning or trigger to determine when more detailed water testing was required.

Table 3.5 Classification of groundwater based on TDS

Classification of groundwater	*TDS (mg/L)*	*Specific conductivity, EC (μS/cm)*
Fresh	<1,000	<1,300–1,700[a]
Brackish	1,000 to 10,000	1,300–1,700 to 13,000–17,000[a]
Saline	10,000 to 100,000	13,000–17,000 to 130,000–170,000[a]

[a] Approximate correlation; precise value depends on ionic composition of water.

Chapter 4

Groundwater effects on the stability of excavations

4.1 INTRODUCTION

To avoid troublesome conditions during excavation and construction, measures must be taken to control groundwater flows and pore water pressures in water-bearing soils. Surface water runoff must also be effectively managed. An understanding of how an excavation may be affected will assist in assessing which groundwater control measures are necessary to ensure stability.

This chapter discusses the various circumstances by which inadequately controlled groundwater could allow unstable ground conditions to develop. The interaction between pore water pressures, effective stress, and stability is introduced. Various large- and small-scale stability problems are presented, and suitable approaches to prevent or control such occurrences are described. The particular groundwater problems associated with the presence of surface water and with excavations in rock are also discussed. The various methods of groundwater control available to stabilize excavations will be outlined in Chapter 5.

4.2 GROUNDWATER CONTROL—THE OBJECTIVES

The fundamental requirement of groundwater control measures is that they should ensure stable and workable conditions throughout so that excavation and construction can take place economically and under safe conditions at all times.

Groundwater may be controlled by exclusion methods (see Section 5.4), one or more types of pumping systems (see Section 5.5) commonly termed "dewatering," or a combination of pumping plus exclusion techniques. This book is primarily concerned with groundwater control by pumping or dewatering, although techniques used to form cutoff barriers for groundwater exclusion schemes are discussed (see Chapter 12).

A correctly designed, installed, and operated dewatering system ensures that construction work can be executed safely and economically by

1. Local lowering of the water table or groundwater level and interception of any seepages due to perched water tables that might otherwise emerge on the exposed slopes or at the base of the excavation(s).
2. Improving the stability of the excavation slopes and preventing material being removed from them by erosion due to seepage. Effective control of groundwater may allow slopes to be steepened and the area of excavation reduced.
3. Preventing base failure of the excavation. Base failure can take several forms, including heaves, blow-outs, or the development of quick conditions in the floor of the excavation. All forms of base failure will have detrimental effects on the bearing capacity at formation level.
4. Draining the soil to improve excavation, haulage, trafficking, and other characteristics of the soils involved. This may allow the use of excavated soil for some purposes that would not be possible if it was dug in a wet "undewatered" condition.
5. Reducing the lateral loads on temporary support systems such as sheet-piling.

The practical upshot of objective 1 is that, by lowering the groundwater level and controlling any local seepages, the excavation will not be flooded by groundwater and might be said to be "dewatered." This is, perhaps, the most obvious aim of groundwater control, i.e., prevention of flooding, but objectives 2 through 5 can be equally important in improving the stability of excavations. This is discussed further in the following sections.

The reader may note that the word "dry" does not appear anywhere in the objectives of groundwater control outlined above. The authors are not in favor of any specifications or contracts that state that the aim of dewatering is to provide dry conditions. After all, a heavy rain shower may cause "wet" conditions, irrespective of how well groundwater is controlled. A much more useful target is for the dewatering measures to provide "workable conditions."

4.3 GROUNDWATER, EFFECTIVE STRESS, AND INSTABILITY

The concept of effective stress is fundamental to understanding the interrelation between groundwater and soil strength and stability. The principle of effective stress was proposed by Karl Terzaghi in the 1920s and is described in detail in soil mechanics texts such as the work of Powrie (2004). As described in Section 3.3, soil has a skeletal structure of solid material with an interconnecting system of pores. The pores may be saturated (wholly filled with water) in confined aquifers or below the water table in unconfined aquifers, or unsaturated (filled with a mixture of water and air) above the water table.

As a saturated soil mass is loaded, the total stress is carried by both the soil skeleton and the pore water. The pore pressure acts with equal intensity in all directions. The stress carried by the soil skeleton alone (by interparticle friction) is, thus, the difference between the total applied stress σ and the water pressure set up in the pores u. This is called the effective stress σ' and is expressed by Terzaghi's equation as

$$\sigma' = \sigma - u \tag{4.1}$$

Because water has no significant shear strength, the soil skeleton may deform while the pore water is displaced (a process known as pore water pressure dissipation). This action continues until the resistance of the soil structure is in equilibrium with the external forces. The rate of dissipation of the pore water will be dependent on the permeability of the soil mass and the physical drainage conditions.

The shear strength τ of soil is primarily from interparticle friction and, therefore, is dependent on the effective stress. The shear strength at failure τ_f can be expressed by the following Mohr–Coulomb failure criterion:

$$\tau_f = \sigma' \tan \phi' \tag{4.2}$$

where ϕ' is the angle of shearing resistance of the soil.

These two simple equations show that reducing pore water pressures (as a result of drawing down the groundwater level) will increase the effective stress within the soil in the area affected. This produces a corresponding increase in the shear strength of the soil, which will improve the stability of the soils around and beneath an excavation.

The increase in effective stresses that results from groundwater lowering is also an important factor in the potential for ground settlements around dewatering systems. This is discussed in Section 15.4.

4.4 LARGE-SCALE INSTABILITY CAUSED BY GROUNDWATER

Inadequately controlled groundwater can cause large-scale stability problems in a variety of ways; the mechanisms likely to be prevalent at a given site and, for a given excavation geometry, will be largely controlled by ground conditions. These large-scale mechanisms can be explained in terms of effective stress and pore water pressures.

Two classic cases of excavation instability are worth considering: (1) groundwater-induced slope instability in an excavation with battered side slopes made into a sandy soil forming an unconfined aquifer and (2) base instability of an excavation caused by the presence of uncontrolled groundwater. The mechanisms of potential instability will be quite different in each case.

4.5 SLOPE INSTABILITY

Consider an excavation with sloping sides dug in a bed of silty fine sand below the standing groundwater level. If the inflow water is pumped from a sump within the excavation, the sides will slump in when a depth of about 0.5–1.0 m below the original standing water level is reached. As digging proceeds, the situation will progressively get worse, and the edges of the excavation will recede. The bottom will soon fill with a sand slurry in an almost liquid condition, which will be constantly renewed by material slumping from the side slopes. The collapse of the side slopes results from the presence of positive pore water and seepage pressures, which are developed in the ground by the flow of water to the pumping sump of the open excavation (Figure 4.1).

The mechanisms causing this unstable condition can be explained in terms of effective stress.

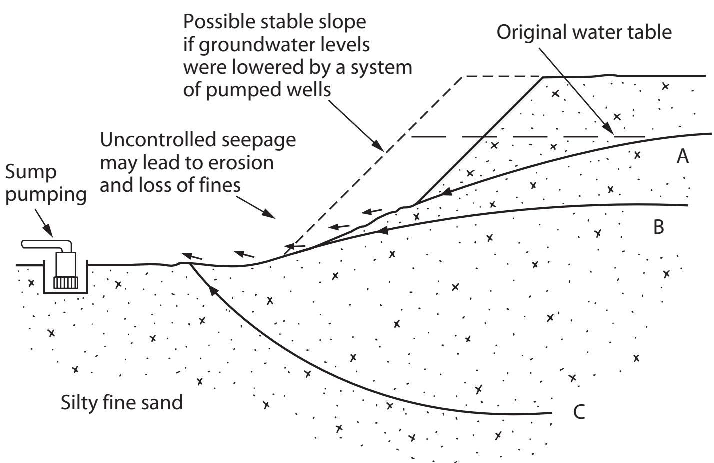


Figure 4.1 Instability due to seepage into an excavation in an unconfined aquifer.

Above the water table, assuming that the soil is dry (i.e., has zero pore water pressure) it can stand in stable slopes of up to ϕ' to the horizontal. In reality, the pore water pressure above the water table may not be zero; in fine-grained soils such as silty sands, negative pore water pressures may exist temporarily due to capillary effects. As a result, stable slopes even steeper than ϕ' may temporarily be possible. However, in time, the negative pore pressure will decay as the capillary water exposed on and near the face dries out. After a finite period (which may be no more than a few hours), the oversteepened slope will eventually crumble to that of the long-term stable slope angle.

Where seepage emerges from the slope, there will be positive pore water pressures. Positive pore water pressures will reduce effective stress, the shear strength of the soil will reduce in turn, and the soil will not stand at slopes as steep as in dry soils. This is the mechanism leading to the "slumping" effect seen when excavating below the groundwater level.

Considering Figure 4.1 in detail, flow line "A" represents the flow line formed by the water table or phreatic surface. Seepage into the excavation will cause a slight lowering of the water table; therefore, flow line "A" curves downward and emerges almost parallel to the surface of the excavation. Immediately below its point of emergence, the positive pore water pressures generated by the seepage mean that the soil can no longer support a slope of ϕ'. Below the emergence of the seepage line, the soil slope will slump to form shallower angles. At the point of emergence of the almost horizontal flow line "B," the sand will stand at ½ ϕ' or less. Where there is upward seepage into the excavation base (flow line "C"), the effective stress may approach zero, and the soil cannot sustain any slope at all. In these circumstances, the soil may "boil" or "fluidize" and lose its ability to support anything placed on it—this is the so-called "quicksand" case.

"Running sand" is another term used to describe conditions when a granular soil becomes so weak that it cannot support any slope or cut face and becomes an almost-liquid slurry. The term is often used as if it were a property of the sand itself. In fact, it is the flow of groundwater through the soil and the resulting low values of effective stress that cause this condition. Effective groundwater lowering can change running sand into a stable and workable material.

In addition to the loss of strength, seepage of groundwater through slopes may cause erosion and undermining of the excavation slopes. This is a problem, especially in fine-grained sandy soils, and is discussed in Section 4.7.

Any solution to groundwater-induced slope stability problems will need to reduce pore water pressures in the vicinity of the excavation. The most commonly used expedient is to install a system of dewatering wells (see

Section 5.5) around the excavation to lower the groundwater level to below the base of the excavation and to ensure that seepage does not emerge from the side slopes. A typical target is to lower the groundwater level a short distance (i.e., 0.5 m) below the deepest excavation formation level.

If the excavation is only penetrating a short distance (i.e., <1.5 m) below the original groundwater level, a sump pumping system (Chapter 8), used in combination with slope drainage (see Section 4.7), might also be effective. Extreme care must be taken when using sump pumping in this way to avoid destabilizing seepages into the excavation; the risk of this increases for excavations further below the original groundwater level.

4.6 BASE INSTABILITY

If the sides of the excavation are supported by physical cutoff walls (see Section 5.4), then the risk of instability of side slopes is not normally a concern. However, the risk of base instability remains.

There are several potential mechanisms of base instability, and the terminology used can be confusing. Eurocode 7 (BS EN 1997-1:2004) identifies four key mechanisms of base instability (these are called Hydraulic Failure in Eurocode 7):

1. Buoyancy uplift (Figure 4.2a)
2. Fluidization due to upward seepage gradients (Figure 4.2b)
3. Internal erosion
4. Piping (Figure 4.2c)

These four types of possible base instabilities caused by uncontrolled or poorly controlled groundwater have been categorized based on the physical processes involved in disrupting or disturbing the soils at the base of an excavation. It should be noted that a wide range of terminologies are used by practitioners to describe the types of failure, e.g., “heave,” “boil,” or “blow.” These terms, although evocative and descriptive of what is observed in the field, are imprecise and do not necessarily allow the instability mechanism to be clearly identified.

The characteristics of each type of base instability are described in the following sections. One thing that all the instability mechanisms have in common is that they will disturb, loosen, or soften the soils at or below the excavation formation level and will have a detrimental effect on the bearing capacity for the structure to be placed in the excavation. Groundwater control measures should be designed and implemented to prevent base instability.

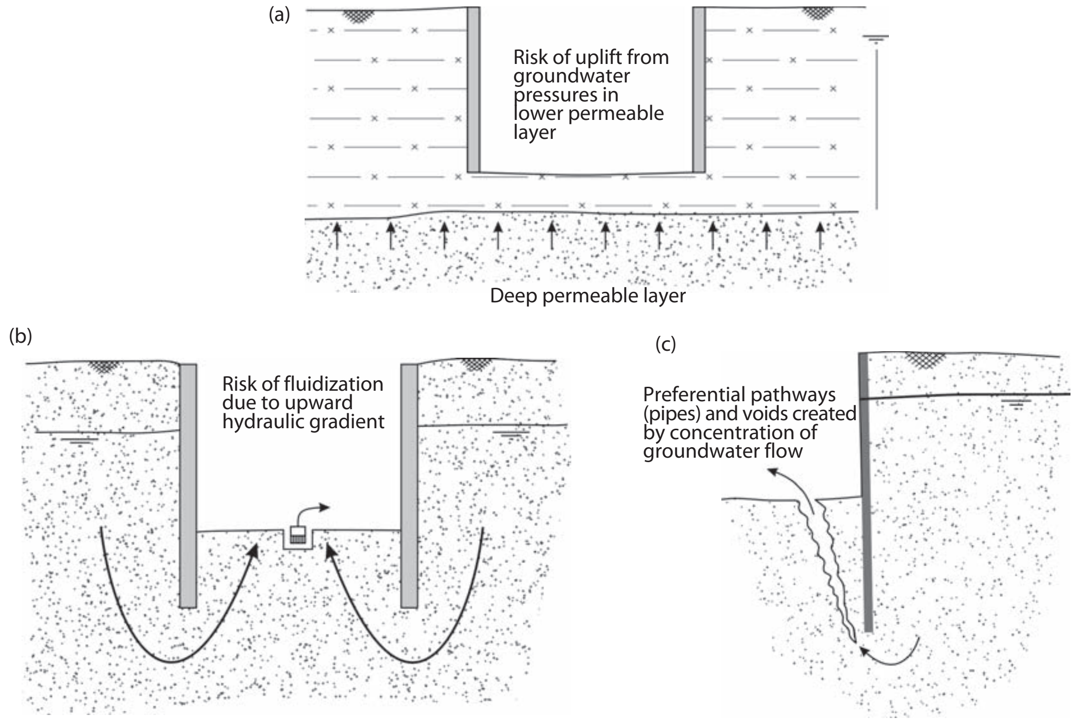


Figure 4.2 Base instability of excavations. (a) Buoyancy or uplift failure. Upward groundwater pressure from the deep permeable layer exceeds the deadweight and shear resistance of the plug of low-permeability soil in the base of the excavation. Heave or uplift may result. (b) Fluidization due to upward seepage gradients. Upward groundwater flow into the base of an excavation results in very low effective stresses. "Boils" or quicksand conditions may result. (c) Piping. Groundwater flow results in the movement of particles within the soil, creating voids or "pipes" within which the flow of groundwater is concentrated.

4.6.1 Base instability by buoyancy uplift

Buoyancy uplift is a risk when the excavation base bottoms out in a layer of low-permeability soil (e.g., clay) or unfissured rock and where there is a permeable stratum containing high pore water pressures at a shallow depth below the excavation formation level. If the weight of the "plug" of low-permeability soil or rock beneath the excavation floor does not significantly exceed the upward force from the water pressures in the deep permeable stratum, buoyancy forces may cause instability of the excavation base.

The classic case in which buoyancy uplift is of concern is shown in Figure 4.3. It shows an excavation above a confined aquifer. The excavation is dug into a clay stratum, with the sides of the excavation supported by walls of some sort. The formation level of the excavation is in a very low permeability clay stratum that forms a confining layer above a permeable confined aquifer. The piezometric level in the aquifer is considerably above the base of the clay layer.

When assessing the stability of this case, the critical horizon is the interface between the confined aquifer and the underside of the overlying confining bed. Consider the two separate sets of pressures acting at this level. The excavation will be stable and safe, provided that the downward pressure from the weight of the residual plug of unexcavated clay is sufficiently greater than the upward pressure of pore water confined in the aquifer. This is assuming that the confining stratum is consistently of very

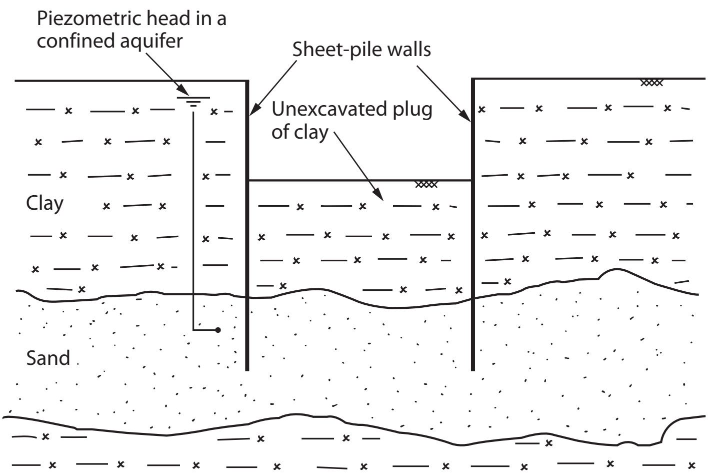


Figure 4.3 Buoyancy uplift failure of excavation in very low-permeability soil overlying a confined aquifer—stable condition. For a relatively shallow excavation, the weight of the unexcavated plug of clay is greater than the upward water pressure from the confined aquifer.

low-permeability and is competent and unpunctured (e.g., it is not penetrated by any poorly sealed investigation boreholes that could form water pathways).

If excavation within the cofferdam is sunk further into the clay without any reduction of pore pressures in the sand aquifer, there will come a time when the clay plug is "buoyant." In other words, the upward water pressure in the aquifer will exceed the downward forces, keeping the clay plug in place (the weight of the clay plug, plus any contribution from the strength of the clay to resist deformation). There will then be an upward movement or "heave" of the excavation formation. If the clay plug heaves sufficiently, the clay may rupture, allowing an uprush of water and sand (sometimes known as a "blow") from the aquifer, which may even lead to the collapse of the excavation (see Figure 4.4).

Instability by buoyancy uplift can be avoided either by

1. Adequately reducing the pore water pressure in the confined aquifer by pumping from suitable dewatering wells inside the cofferdam. Alternatively, the wells may be sited outside the cofferdam but will be less hydraulically efficient. The wells should lower the aquifer water pressure so that the downward forces exceed the upward pressures by a suitable factor of safety.
2. Increasing the depth of the physical cutoff wall sufficiently to penetrate below the base of the confined aquifer. Provided that the cutoff

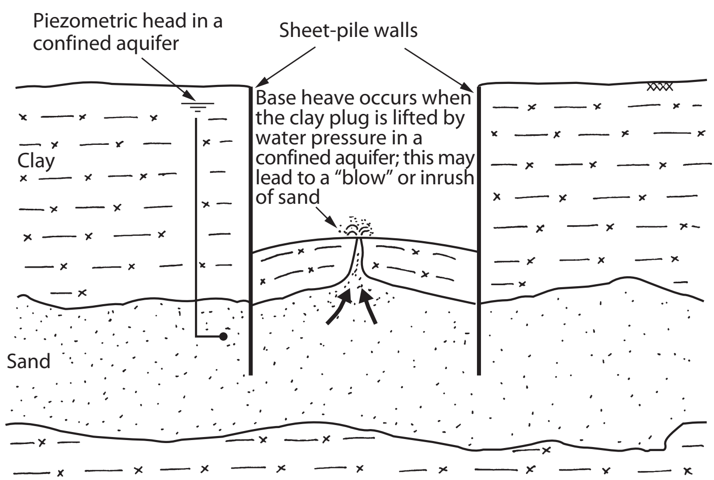


Figure 4.4 Buoyancy uplift failure of excavation in very low-permeability soil overlying a confined aquifer—unstable condition. For a deeper excavation, the upward water pressure from the confined aquifer exceeds the weight of the clay plug, leading to base heave and, ultimately, a "blow."

walls are watertight, this will prevent further recharge, thus leaving only the water pressure contained in the aquifer within the cutoff walls to be dealt with. This can be affected by installing relief wells (see Section 11.5) before excavation. The economics of this alternative will depend primarily on the depth needed to secure a seal and the effectiveness of that seal.

3. Increasing the downward pressure on the base of the excavation by keeping it partly topped up with water during the deeper stages of work. Excavation is made underwater, and a trémie concrete plug is used to seal the base on completion.

This describes the principal features of the "buoyancy uplift" instability in confined aquifers. For relatively narrow excavations, the shear resistance between the clay plug and surrounding ground may be significant and should be considered when estimating factors of safety. Hartwell and Nisbet (1987) discuss the problem further.

Buoyancy uplift can also sometimes occur in nominally unconfined aquifers that contain discrete (but perhaps very thin), very low-permeability clay layers. Figure 4.5 shows a case in which dewatering wells are used to lower groundwater levels around an excavation but do not penetrate the clay layer below the excavation. The dewatering system has lowered water pressures above the clay layer, but the original high pressures remain

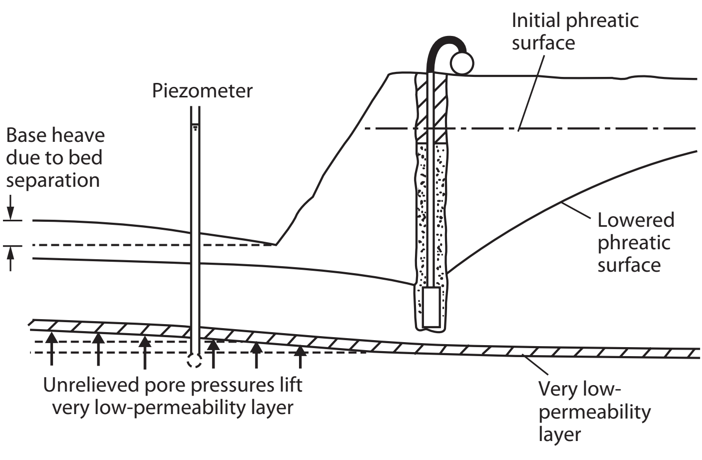


Figure 4.5 Pore water pressures not controlled beneath thin very low-permeability layer—unstable condition. (From Preene M., and Powrie W., Construction dewatering in low-permeability soils: Some problems and solutions. *Proceedings of the Institution of Civil Engineers, Geotechnical Engineering*, 107, 17–26, 1994. Reproduced by permission of ICE Publishing.)

beneath. These high pressures can cause uplift. This mechanism is sometimes known as "bed separation," because as the clay layer moves upward, a reservoir of water will develop underneath. This form of uplift can be avoided by using deeper dewatering wells to control pore water pressures at depth.

4.6.2 Base instability by fluidization due to upward seepage gradients

Where an excavation is made through a thick permeable stratum (such as sand and gravel), groundwater control requirements can be reduced if the sides of the excavation are supported by physical cutoff walls. The cutoff walls remove the need to consider instability of side slopes, and particularly for relatively small excavations such as shafts or cofferdams, it may be possible to lower the groundwater levels simply by sump pumping from within the excavation. The methods used to form cutoff walls are described in Chapter 12.

However, when pumping from within an excavation contained within a cutoff wall, there is a risk that upward hydraulic gradients due to pumping will significantly reduce effective stresses in the soil. In extreme cases, effective stresses will approach zero, and the soil will fluidize and lose all strength. This could lead to catastrophic structural failure of the excavation and associated cutoff wall.

The risk of fluidization of the base due to upward seepage is related to the upward hydraulic gradient generated by pumping (Figure 4.6). Fluidization

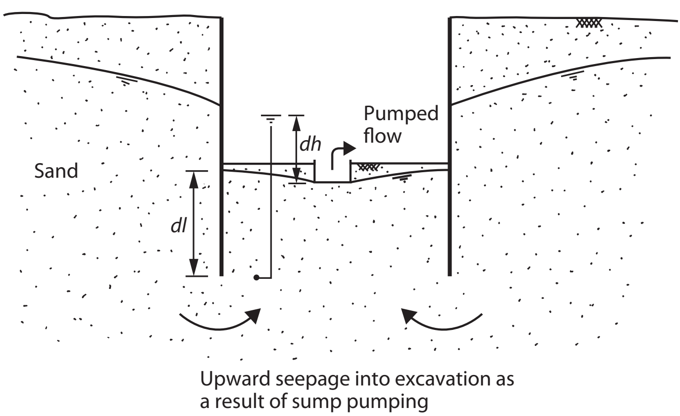


Figure 4.6 Fluidization of excavation base due to upward seepage gradients in an unconfined aquifer. Upward hydraulic gradient below excavation floor = *dh*/*dl*.

will theoretically occur when the upward hydraulic gradient exceeds a critical value i_{crit}

$$i_{crit} = \frac{(\gamma_s - \gamma_w)}{\gamma_w} \tag{4.3}$$

where γ_s is the unit weight of soil, and γ_w is the unit weight of water. In general, the density of soil is approximately twice the density of water (peat being a notable exception); therefore, in general, $i_{crit} \approx 1$. This is the hydraulic gradient at which fluidization will occur, and thus, it is important that this value is not approached. In design, it is normal to limit the predicted hydraulic gradients to be significantly less than i_{crit} to provide a suitable factor of safety.

The hydraulic gradient for upward seepage is defined as the head difference *dh* divided by the flow path length *dl* (see Figure 4.6). The hydraulic gradient can be controlled in two ways:

1. By increasing *dl*. This can be achieved by ensuring that the cutoff walls penetrate a sufficient depth below formation level. This is an important part of the design of cutoff walls in unconfined aquifers (see the work of Williams and Waite 1993).
2. By reducing *dh*. This requires reducing the groundwater head difference between the inside and the outside of the excavation enclosed within the cutoff walls. The most obvious way of doing this is to use a system of dewatering wells outside the excavation to lower the external groundwater level and, therefore, reduce the head difference to an acceptable level while still sump pumping from within the excavation. This approach might be taken a step further, and external wells can be used to lower the groundwater level to below the formation level and avoid the need for sump pumping altogether. An alternative approach to reducing *dh* is to keep the excavation partly or fully topped up with water and carry out excavation underwater (for example, using grabs) and then place a trémie concrete plug in the base. This method is sometimes used for the construction of shafts and cofferdams.

Alternatively, a system of relief wells (see Section 11.5) could be installed within the excavation to provide preferential, engineered flow paths for groundwater to enter the excavation. This will dramatically reduce the hydraulic gradients within the excavation and, provided the relief wells are spaced closely enough apart, will prevent fluidization of soil in the base of the excavation.

4.6.3 Base instability by internal erosion

Internal erosion involves the movement of soil particles under the influence of flowing groundwater. Although Eurocode 7 (BS EN 1997-1:2004) identifies internal erosion as a separate mode of instability, in practice, it is often associated with instability caused by fluidization due to upward seepage gradients (Figure 4.2b).

Internal erosion may occur where hydraulic gradients are high (for example, when groundwater flow is concentrated where it enters the excavation or flows into a sump). Soils that are most prone to erosion are uncemented granular soils which are uniformly graded (in which the majority of the soil particles are within a narrow size range) or are internally unstable (see the work of Skempton and Brogan 1994), in which the finer particles can move within a skeleton of larger particles. The removal of fine particles (termed "loss of fines") can result in loosening of the soils and the creation of voids, which may collapse. Loss of fines is a particular risk where sump pumping is used (see Section 8.7).

The measures used to control hydraulic gradients and reduce the risk of base instability from fluidization due to upward seepage gradients (see Section 4.6.2) are generally also suitable for use to reduce the risk of internal erosion at the base of an excavation. Additionally, the risk of erosion of fine soil particles from the zones where groundwater is pumped within the excavation can be reduced by using sumps with adequate filters around them, or by replacing sump pumping with wellpoints or deep wells equipped with suitable filter media.

4.6.4 Base instability by piping

Although identified in Eurocode 7 (BS EN 1997-1:2004) as a separate mode of instability, piping is most commonly the result of instability caused by fluidization due to upward seepage gradients (Figure 4.2b) in combination with internal erosion. In these circumstances, high hydraulic gradients and potentially mobile granular soils can result in the washing out of fine particles from the soil to create preferential flow paths. Water will naturally tend to take the easiest path; thus, water flow is further concentrated in these zones, causing further erosion of soils. This can continue until most of the water is entering the excavation through a small number of discrete open voids, termed "pipes" created by the washing out of fine particles along the preferential flow paths (Figure 4.2c). Where piping channels have occurred, it is likely that inflows to the excavation will be significantly increased compared with nonpiping conditions. Piping channels are often highly unstable and may collapse unexpectedly, threatening the stability of the excavation and support structures.

The measures used to control hydraulic gradients and reduce the risk of base instability from fluidization due to upward seepage gradients (see Section 4.6.2) are generally also suitable for use to reduce the risk of piping instability in the base of an excavation.

4.7 LOCALIZED GROUNDWATER PROBLEMS

In addition to groundwater effects that can cause large-scale failure of an excavation, groundwater can also cause more localized problems in excavations. The effects of localized problems can vary greatly in scope, from minor inconveniences caused by a perched water table to severe problems resulting from the collapse of side slopes due to spring sapping; localized groundwater problems should not be ignored. A number of possible mechanisms are described in the following sections.

4.7.1 Drainage of slopes and formation

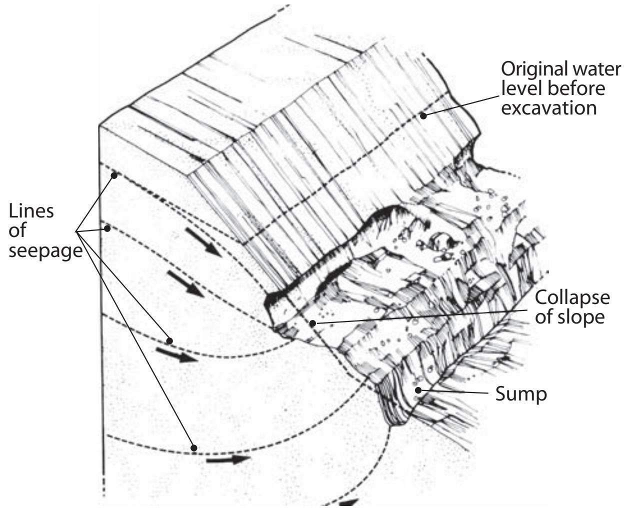
The slopes of open-cut excavations should be designed so that instability due to seepage and associated continuous removal of fines does not occur. Where the slope is too steep or the hydraulic head too large, seepage can daylight on the slope and cause slope failure (see Figure 4.7). The term daylight is another expression for local trouble spots, but they may not be only local.

Oftentimes, the solution is to have a flatter slope, sometimes together with a toe drain backfilled with filter media (see Figure 4.8), to prevent emerging water from continuously removing fines, therefore causing damage leading to instability. It is advantageous to return the filter material back up the slope a short distance and, thus, prevent seepage in that area from transporting fines. Periodic maintenance may be needed to ensure that toe drains remain efficient.

An alternative solution is to adopt a slightly steeper slope angle with a properly backfilled drainage trench at the toe of the slope, in conjunction with sandbagging of the toe of the slope or weighting with graded filter material. Alternatively, the fill can be weighted with loose-laid timbers and sacking, with straw or hay padded behind them. Modern practice also allows the use of one of the various proprietary geotextile fabrics. Sandbagging or a geotextile membrane may be needed where local perched water tables are revealed. The materials to be used will depend on local availability in the country concerned.

The gradients within the excavated floor areas should be formed so that the runoff water is directed to the collector drains and the risk of ponding is assiduously avoided. Runoff from slopes should be collected by the surface water drainage system, suitably sited at each berm or toe of slope, and thence be directed to pumping sumps for ultimate disposal off site.

(a)



(b)

Figure 4.7 Effect of seepage on excavation. (a) Collapse of excavation caused by seepage from steep slopes. (From Somerville, S.H., Control of groundwater for temporary works. *CIRIA Report 113*, Construction Industry Research and Information Association, London, 1986. Reproduced by kind permission of CIRIA: www.ciria.org.) (b) Unstable excavation slope resulting from seepage. Seepage has eaten back into the slope, and an outwash fan of saturated washed-out material has formed at the base of the slope.

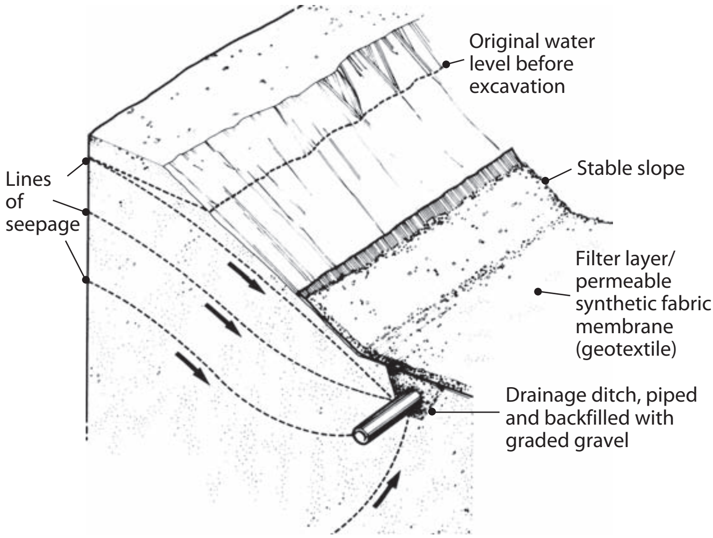


Figure 4.8 Stable excavated slopes resulting from flat gradient and provision of drainage trench. (From Somerville, S.H., Control of groundwater for temporary works. *CIRIA Report 113*, Construction Industry Research and Information Association, London, 1986. Reproduced by kind permission of CIRIA: www.ciria.org.)

Sumps are usually sited at the corners of excavations, below the general excavation level, and made big enough to hold sufficient water for pumping and to keep the excavation floor relatively dry. A pump is provided for each sump and connected to a discharge pipe.

The drains leading to the sumps should be so arranged as to allow drainage of the whole excavation and have sufficient fall to prevent silting up. Also, the drains should be cleaned out from time to time. Ditches should be sufficiently wide to allow a water velocity low enough to prevent erosion. This may be achieved by constructing check weirs at intervals along the ditch. Additionally, the ditches can be improved by laying rough blinding or paving material or laying porous open-jointed pipes or other agricultural land drainage piping surrounded by filter material.

The observance of good practice applicable to slope grades and filter design criteria will help achieve stable slopes but it is essential to be on the lookout for local trouble spots, generally due to variations in soil conditions.

4.7.2 Perched water table

Often within permeable granular strata, random lenses of significantly lower permeability occur, locally inhibiting downward drainage of water. A "perched water table" is likely to result from this phenomenon (Figure 4.9). This means

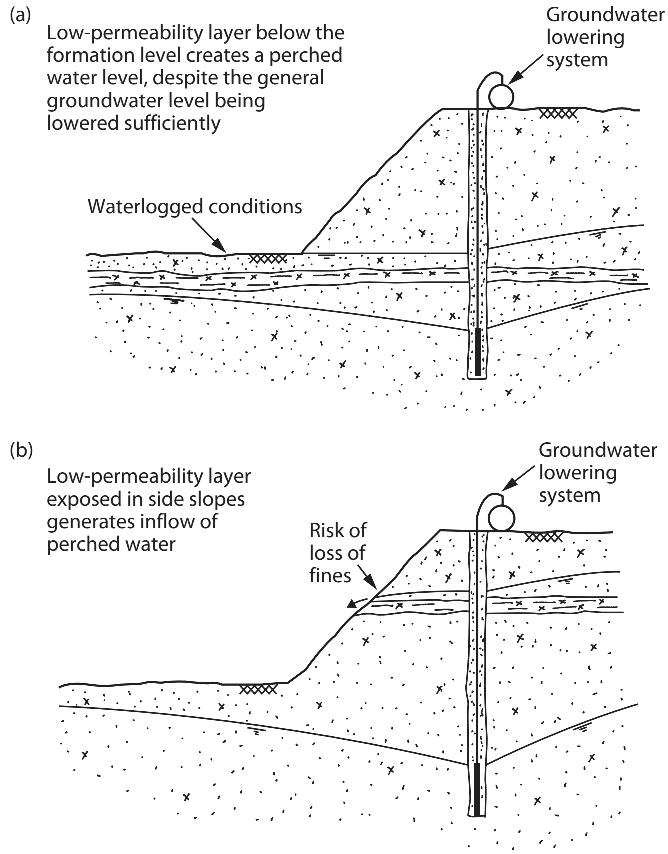


Figure 4.9 Perched water table. (a) Low-permeability layer below the formation level. (b) Low-permeability layer above the formation level and exposed in the side slope.

that a residual body of water remains just above the lens of low-permeability soil, even though the groundwater level in the permeable strata beneath has been drawn down lower. This troublesome condition can be dealt with effectively by forming vertical drainage holes through the low-permeability lens. This can sometimes be done by the simple expedient of digging through the layer with an excavator bucket. Alternatively, relatively small-diameter sand drains may be appropriate, perhaps jetted in using wellpoint equipment (see Section 9.8).

If seepage from perched water tables continues once the excavation has passed through the perched water table, this is known as residual seepage or "overbleed." If the rates of seepage are low, the water will merely constitute a nuisance rather than lead to a serious risk of flooding. Nevertheless,

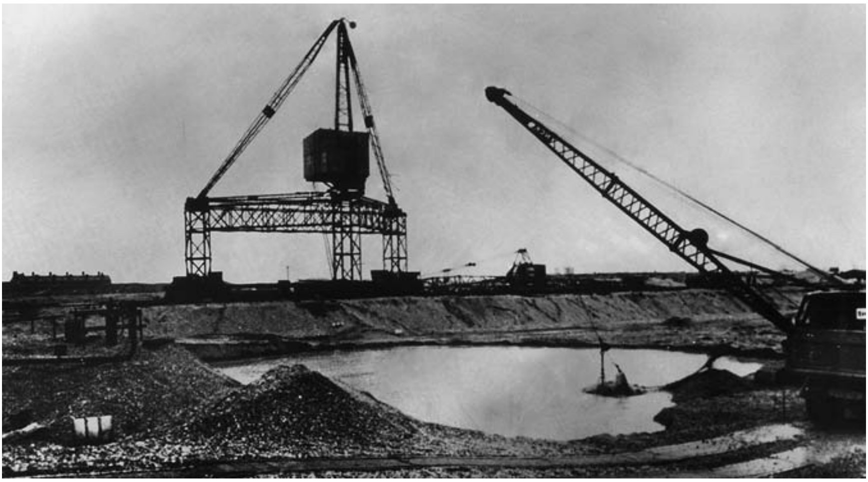


Figure 4.10 Perched water table. The groundwater level monitored in observation wells was several meters below the standing water visible in the photograph. When the clay layers below this level were punctured by excavating, the perched water drained away. (Courtesy of ESG Soil Mechanics, Burton on Trent, U.K.)

even at small rates of seepage, if fines are being transported by the water, this should be counteracted immediately. Possible measures include the placement of sand bags, a granular drainage blanket, or geotextile mesh to act as filters.

An extensive perched water table in an unconfined aquifer of gravelly sand is shown in Figure 4.10. Readings in several observation wells indicated that the pumping from a system of deep wells had drawn the groundwater level down to the target level, which was several meters below the perched water level visible in the figure. The contractor was advised to continue excavation by dragline at one location. Within a few hours of continuous digging, the water started to swirl and there was a loud gurgling sound as the perched water drained away.

4.7.3 High-permeability zones

Sometimes, a lens of significantly more permeable water-bearing soil, not previously detected, may be revealed during excavation. This will act as a source of copious recharge requiring additional pumping capacity to be installed to abstract groundwater from the lens. Such permeable gravel lenses are not uncommon in fluvio-glacial or alluvial soils that were laid down in water. The permeable zones may result from paths of former streams or flow channels within the geological deposits. Like modern-day watercourses, these gravel lenses may follow an irregular, difficult-to-predict path. Long, thin gravel lenses are known as "shoestrings"; these may easily pass undetected during site investigation.

The same recharge effect as from a natural gravel lens can result from man-made features such as the permeable gravel bedding or subdrains associated with sewers (see Section 15.5.3). There have been cases in which dewatering adjacent to existing sewers has proved difficult, until the flow path formed by the permeable material around the sewer was blocked, or the flow was intercepted directly by sump pumping from the gravel bedding.

Figure 4.11 shows a case described by Preene and Powrie (1994), in which an ejector well system for a small shaft excavation lowered the general groundwater level but failed to prevent persistent troublesome seepage on one side of the shaft. Additional ejector wells installed around the shaft (and geologically logged as site investigation boreholes) revealed the presence of a shoestring of permeable gravel present within the main stratum of fluvio-glacial sand.

Additional wells were installed into the shoestring to intercept the water before it reached the excavation, the seepage was eliminated, and the shaft successfully completed. This sounds very straightforward, but this was far from the actual story. Because the location of the shoestring was not known precisely, the dewatering contractor had to keep drilling wells until a few of them directly intercepted the permeable zone. Up until that time, even when many additional wells had been installed and pumped, the seepage

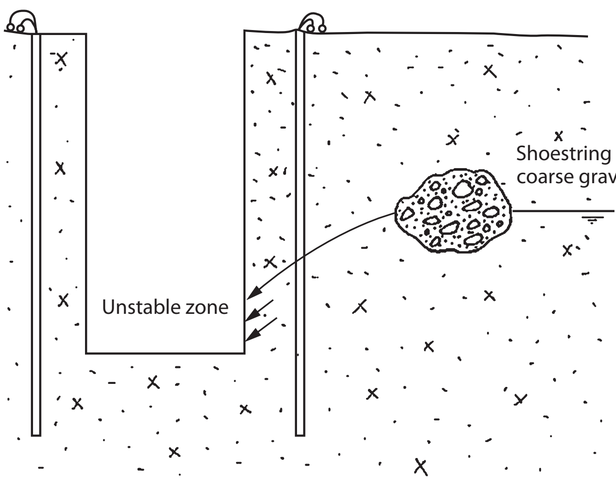


Figure 4.11 Effect of close source of recharge on effectiveness of groundwater lowering for shaft excavation. A permeable gravel shoestring acts as a close source of recharge and concentrates seepage on one side of the shaft, leading to local instability. (From Preene M., and Powrie W., Construction dewatering in low-permeability soils: Some problems and solutions. *Proceedings of the Institution of Civil Engineers, Geotechnical Engineering*, 107, 17–26, 1994. Reproduced by permission of ICE Publishing.)

persisted. The total number of wells increased from five to 22. In reality, only the handful of wells pumping directly from the shoestring were truly effective.

The principle to be remembered is that, whenever a local source of more copious recharge is encountered, additional wells or sumps should ideally draw groundwater direct from the recharge source (i.e., the permeable zone). It is preferable to intercept the flow upstream of the exposed excavation, but sometimes, this may not be practicable. In such circumstances, additional wells or sumps must be installed as close as possible to the area of water emergence into the work.

4.7.4 Spring sapping and internal erosion

This mechanism, which can occur in sands and silty fine sands, is not uncommon but is not well understood. The sides of an excavation in silty fine sand may show gully structures from which sand slurry can be seen to flow into the bottom of the excavation. This is due to internal erosion. The water flowing along the upper surface of the lowered phreatic line carries sand slurry with it and thus tunnels backward beneath the overlying sand, which has some strength due to capillary tension. Before this tunneling effect has gone far, the top collapses, and the collapsed material is also carried away. This tunneling phenomenon has often been observed in fine sands, especially in silty fine sands. It tends to occur immediately above a thin and less permeable layer of finer material that has caused a perched water table, although this condition may only be temporary. This phenomenon is quite well known in geology as spring sapping or seepage erosion.

In the 1960s, an interesting experiment into this phenomenon was carried out at Imperial College, London. A slope of fine sand was formed in a glass-sided tank, and a small source of water was allowed to flow in through the base of the tank. The base of the tank was at a flat angle. A very small flow of water was introduced beneath the sand and erosion rapidly started at the point of exit. One would intuitively expect a cone of eroded material to build up and for the process of erosion to come to an end, but in fact, the flow of sand was continuous, as it was in a case described by Ward (1948). It seems that this flow can only happen if excess pore pressures exist in eroded material, but their cause is still unexplained, and it is hard to see why they are not quickly dissipated.

The site studied by Ward (1948) was near Newhaven, Sussex, at a point where Woolwich and Reading Beds (now known more correctly as the Lambeth Group) overlie the Thanet Sand and Chalk. A thin bed of fine sand in the Woolwich and Reading Beds carried a small flow of groundwater toward the sea, and although the hydraulic gradient was only about one-in-ten, internal erosion on a large scale had continued for centuries, and debris flows remained active and carried the sand and the collapsed mass of the overlying clay down to the beach some 55 m below. This state of affairs was eventually controlled by a simple system of filters and drains.

The curious thing about this process is that it takes place when the flow of water is very small and the gradient was very low, yet because it is continuous, in time, very large quantities of material are removed.

Figure 4.12 shows a close-up view of a small example of spring sapping. A thin lens of predominantly shells was present within an extensive stratum of fine sand. The levels recorded in nearby observation wells indicated that the water had been drawn down to some 4 m below this lens, but because the permeability of the lens of shelly material was significantly greater, there was still water flowing, continuously transporting fines and, therefore, causing local spring sapping. This was evidenced by the two cavities at the center of the photograph and the outwash fans of almost liquid sand in the foreground.

Variable soil conditions may be the cause of local trouble spots being revealed as excavation proceeds. Drift deposits are often heterogeneous, and therefore, the possibility of exposing local trouble spots in the sides of excavations is ever present. Oftentimes, these trouble spots are due to a layer or lens of slightly lower permeability material within the stratum being dewatered. Because of the difference in permeability, the pore pressures in the layer or lens are higher than in the surrounding stratum, and despite the general effectiveness of the installed groundwater lowering system, there is some residual flow from the lens, which causes transportation of fines. If this condition is allowed to persist, it will result in instability due to seepage erosion.

Figure 4.12 Spring sapping. Local spring sapping has caused the two cavities at the center of the photograph with outwash fans of almost liquid sand in the foreground.

4.8 EXCAVATIONS IN ROCK

The preceding sections have described the effect of groundwater on excavations on uncemented soil or "drift" deposits. Groundwater lowering is most commonly used in these material types but can also be used for excavations in "rock." For the purposes of groundwater control, the defining feature of rock is that the rock mass is relatively strong and often of low permeability. As a result, groundwater tends not to flow through the whole rock mass but tends to be concentrated locally along fissures, fractures, joints, solution fissures, and other discontinuities.

In general, groundwater-induced instability is much less of a problem for excavations in rocks compared with excavations in soils. An excavation in rock below the water table will encounter localized inflows where joints or fissures are intercepted. Provided that the rock around the fissures is competent, these inflows may not cause stability problems. In such cases, the aim is to prevent the inflows from affecting construction operations, and sump pumping is often used to manage water within the excavations.

A number of potential problems may result from groundwater inflows into rock excavations:

1. The concentration of flow where water flows from a fissure into the excavation may create high flow velocities and an associated risk of internal erosion. There have been cases in relatively soft mudstones and sandstones in which initial inflows were modest but increased significantly with time as the fissures were enlarged by erosion. One solution would be to install a system of external dewatering wells to reduce the groundwater head driving flow into the excavation. This will reduce the potential for erosion.
2. Weathered beds or zones may exist within otherwise competent rock. These zones may behave like drift deposits and may slump inward as groundwater flows through them. These problems may be avoided by the use of external dewatering wells or, if very localized, by the use of drainage measures similar to those described in Section 4.7.
3. Groundwater will emerge into the excavation at locations dictated by the rock structure, bedding, and fissuring. These locations will often be inconvenient for construction operations; trying to place high-quality structural concrete on top of a gushing fissure is unlikely to be efficient. This problem can be dealt with by constructing trench drains (see Section 5.3) to carry the water away from the working area to a sump. Alternatively, an array of relief wells (see Section 11.5) could be drilled through the base of the excavation to try and intercept fissures and to provide a preferential pathway to allow groundwater to enter the excavation at a more convenient location (Figure 4.13).

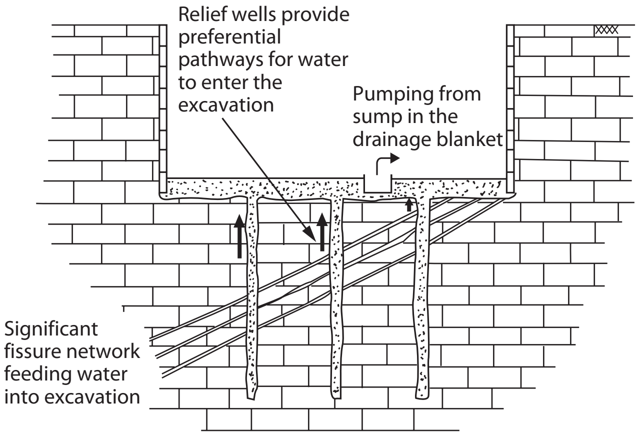


Figure 4.13 Relief wells used in rock excavations to intercept flow in discrete fissures.

4. If the rock possesses bedding that is roughly horizontal, pore water pressures may remain trapped beneath lower permeability layers, leading to the "buoyancy uplift" type problem described in Section 4.6.1. This sometimes occurs where the rock consists of alternating layers of mudstone (relatively impermeable) and sandstone (relatively permeable); such conditions exist in the Coal Measures Rock in Northern England. The installation of relief wells within the excavation will prevent this by allowing the water trapped in the more permeable zones to flow freely into the excavation. Alternatively, pumped deep wells may be used to depressurize specific permeable layers with a layered rock sequence.
5. In very large rock excavations, there may be a risk of "block failure," in which part of the rock mass slides or moves into the excavation, often moving along joints or fissures that slope into the excavation. The pore water pressures acting in the rock joints have a critical effect on the stability of such rock masses. Dewatering wells have occasionally been used to control pore water pressures as part of stabilization measures. This is a very specialized case, and it is essential that rock mechanics specialists are involved if this approach is being contemplated.

4.9 SURFACE WATER PROBLEMS

This book is primarily concerned with the control of groundwater, but because the aim is to provide workable conditions for construction, the importance of surface water control must not be forgotten. Surface water may arise in many ways: as direct precipitation into the excavation, as precipitation runoff from surrounding areas, as leakage through cutoff walls used to exclude groundwater, and as waste water from construction operations such as concreting or even from washing down the plant and equipment. Because the excavation is obviously going to form a low point in the site, surface water, if given free rein, is likely to collect at the bottom of the excavation.

Regardless of the source, if surface water is allowed to pond in the excavation, it will impede efficient excavation and construction. Figure 4.14

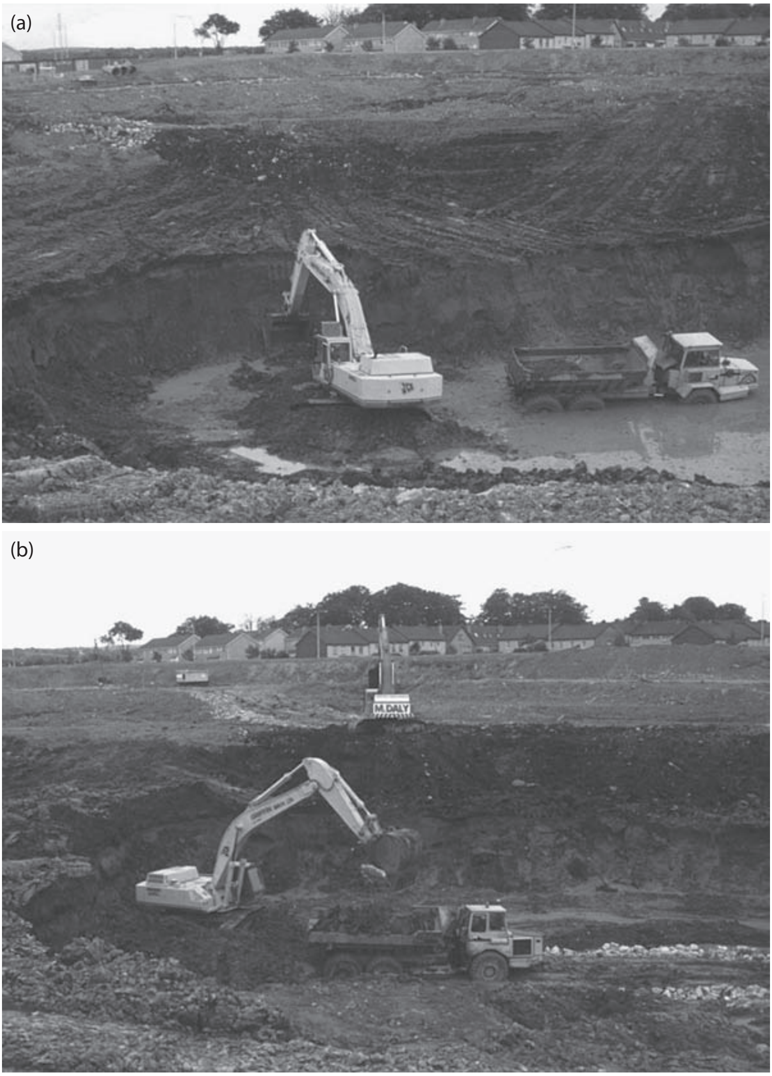


Figure 4.14 Control of surface water in excavations. (a) Without adequate surface water control. (b) With effective surface water control.

shows an example of an excavation before and after surface water was suitably dealt with. Neglecting the management of surface water will undoubtedly cost the project time and money. Methods for control of surface water are described in Section 5.2 and Section 8.3.

4.10 EFFECT OF CLIMATE AND WEATHER

Depending on climatic conditions, it may be prudent to make major or minor contingency plans. For instance, in some parts of the world, it is known that there is a risk that monsoons, typhoons, and hurricanes will occur at certain seasons of the year and the consequential high volume surface runoff may be very significant. Hence, planning for such areas would dictate that appropriate measures to deal with sudden influxes of surface water, prevent slope erosion, and deal with short-term increases in groundwater level must be incorporated into the excavation plan at the design stage. In particular, when working in shallow unconfined aquifers, intense rainfall events can have a major effect on lowered water levels. Figure 4.15 shows groundwater level and rainfall data taken during the rainy season at a site in Southeast Asia. A wellpoint system was used to lower water levels in a very silty sand and achieved a drawdown of several meters. Whenever there was heavy rain, the water levels rose

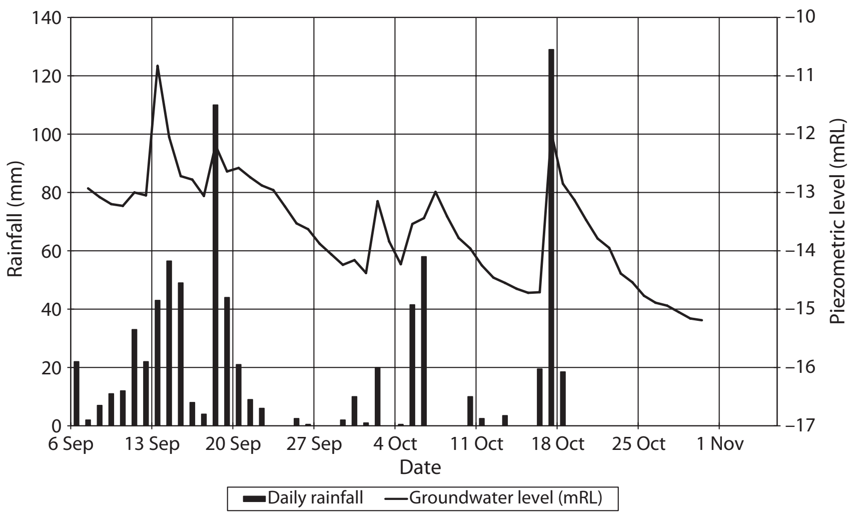


Figure 4.15 Effect of extreme rainfall events on groundwater levels. Data are from monitoring of a wellpoint system operating on a site in Southeast Asia during the rainy season.

rapidly and only fell back to their original, lowered levels slowly under the influence of pumping.

Exposed excavated slopes may be protected from monsoon-type erosion by laying an appropriate geotextile membrane. A judgment should be made at the design stage whether to cater to the one in 10-year occurrence, or the 50-year occurrence or more onerous conditions.

Chapter 5

Methods for control of surface water and groundwater

5.1 INTRODUCTION

As previously discussed in Chapter 4, the presence of groundwater could lead to troublesome conditions when construction operations are to take place below the original groundwater level. Surface water runoff must also be effectively controlled to prevent interference with excavation and construction works. This chapter briefly outlines some of the methods available for the control of surface water and groundwater.

Techniques for the control of groundwater can be divided into two principal types:

1. Those that exclude water from the excavation (known as exclusion techniques)
2. Those that deal with groundwater by pumping (known as dewatering techniques)

This chapter includes brief discussions and tabular summaries of the various techniques available for both groups, including their advantages and disadvantages. This book primarily addresses dewatering techniques. The practicalities of the more commonly used pumping or dewatering methods are described in detail in subsequent chapters. The techniques used for groundwater exclusion are discussed in Chapter 12.

This chapter, and indeed the rest of the book, focuses primarily on groundwater control methods for construction projects. However, the techniques described here can be used in other fields, including the mining industry. The principles of groundwater control by exclusion or pumping are regularly applied to open-pit and underground mines, so the methods outlined in this chapter should also be of interest to mining engineers faced with groundwater problems.

5.2 CONTROL OF SURFACE WATER

To optimize efficiency (and profitability), it is essential that conditions within the area of excavation and construction be workable; mechanical plant should be able to operate efficiently without getting bogged down. To achieve this, in addition to lowering of groundwater levels, surface water must be controlled. Thus, both surface water runoff and groundwater should be disposed of expeditiously.

On many construction sites, the measures enacted to control surface water runoff are inadequate, and result in considerably unnecessary waste of time and money.

There are three fundamental tenets to the efficient control of surface water on construction sites:

1. Source control
2. Water collection
3. Water treatment

5.2.1 Control of surface water runoff

The basic rule of source control is to deal with surface water before it can become a problem. In other words, collect and control the surface water runoff as soon as or, better still, even before it enters the area of work.

If water is allowed to run over exposed soil, it will pick up soil particles which will either be deposited elsewhere on the site, potentially blocking drains and ditches, or will be carried in the water to the discharge point, where it will require treatment to avoid pollution of the receiving watercourse.

The drainage system should incorporate appropriate measures to collect runoff from land areas surrounding and adjacent to an excavation to prevent surface water runoff from encroaching into the construction area. Adoption of this philosophy requires the installation of adequate interceptor drains sited uphill or upstream of the excavation at the original ground level.

Rainwater or discharges from other construction activities (such as concreting or washing down of plant) should be prevented from entering an excavation, particularly on a sloping site, by the simple expedient of digging collector ditches or drains on the high ground. The drains should lead the water away to discharge points (which could be pumped by sump pumps, see Chapter 8) lower down the slope.

A key element in the successful control of surface water is often the education and training of foremen, excavator operators, and other site operatives. If they can be convinced that a little effort in minimizing the generation of surface water runoff, and diverting water away from exposed areas of bare soil, will benefit them by reducing the need to clean out drainage ditches and avoiding pollution at the discharge point, then site works will often go much more smoothly.

5.2.2 Collection of surface water runoff

Some surface water in the construction area is unavoidable, for example, as the result of rainfall into the excavation. This water needs to be collected and carried away, either by gravity flow or by pumping, to a disposal point.

Collector drains should be lined with an impermeable membrane to avoid potential upstream recharge that might cause a rotational slip of the excavation slope. The open ditch method (see Figure 5.1a) should be used only where the presence of open ditches does not inhibit construction work. As an alternative, agricultural type drains can be used (see Figure 5.1b), but the surface must be regularly scarified to reduce the effect of clogging by suspended particles in the surface water. This cleaning dictum should be applied to both open ditch and agricultural type drains.

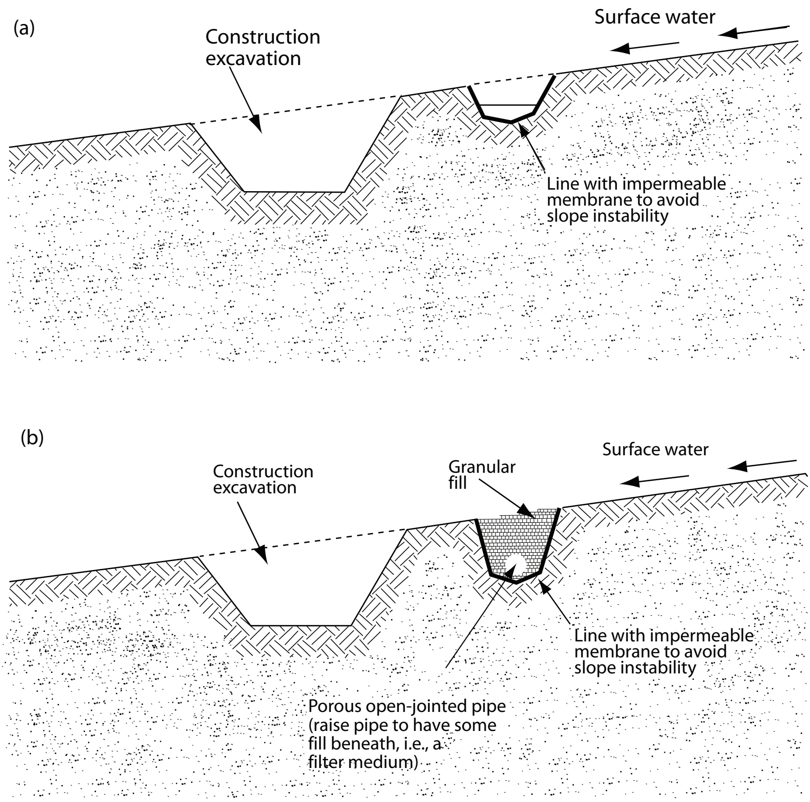


Figure 5.1 Drains for collection of surface water runoff. Open ditch (a); agricultural drain (b). (After Somerville, S. H., Control of groundwater for temporary works. *CIRIA Report 113*, Construction Industry Research and Information Association, London, 1986. Reproduced by kind permission of CIRIA: www.ciria.org.)

5.2.3 Treatment of surface water runoff

Water collected in surface water drainage systems on construction sites will generally have some suspended solids (clay, silt, and sand-sized particles) in it, as a result of its transit across site. On most sites, it is not acceptable to discharge this water from the site without some form of water treatment to reduce suspended solids to acceptably low levels. If significant concentrations of solids are discharged into a watercourse, there is significant potential for environmental and ecological harm (see Section 15.8). Settlement lagoons or more sophisticated water treatment systems may be required (see Section 15.8.2).

5.3 METHODS OF GROUNDWATER CONTROL

The techniques available for the control of groundwater fall into two principal groups:

1. Those that exclude water from the excavation (known as exclusion techniques)
2. Those that deal with groundwater by pumping (known as dewatering techniques)

This book is primarily concerned with the second group, the dewatering methods. However, the essential features of groundwater control by exclusion are outlined in the following section, and potential techniques are discussed in Chapter 12. Situations in which exclusion and pumping methods might be used in combination are discussed in Section 5.7.

5.4 EXCLUSION METHODS

The aim of groundwater control by exclusion is to prevent groundwater from entering the working area. The methods used can be grouped into three broad categories:

1. Methods in which a very low-permeability discrete wall or barrier is physically inserted or constructed in the ground (e.g., sheet-piling and diaphragm walls).
2. Methods which reduce the permeability of the in situ ground (e.g., grouting methods and artificial ground-freezing).
3. Methods which use a fluid pressure in confined chambers such as tunnels to counterbalance groundwater pressures (e.g., compressed air and earth pressure balance tunnel-boring machines).

The techniques used to exclude groundwater are listed in Table 5.1, which is based on information from Preene et al. (2000). Details of the various methods in groups 1 and 2 can be found in Chapter 12. Discussion of group 3 methods (tunneling) can be found in Section 5.6.

One of the most common applications of the exclusion method involves forming a notionally impermeable physical cutoff wall or barrier around the perimeter of the excavation to prevent groundwater from entering the working area. Typically, the cutoff is vertical and penetrates down to a very low-permeability stratum that forms a basal seal for the excavation (Figure 5.2a).

The costs and practicalities of constructing a physical cutoff wall are highly dependent on the depth and nature of any underlying permeable stratum. If a suitable very low-permeability stratum does not exist, or is at great depth, then upward seepage may occur beneath the bottom of the cutoff wall, leading to the risk of base instability (see Section 4.6). In such circumstances, dewatering methods may be used in combination with exclusion methods (Figure 5.2b). Alternatively, it may be possible to form a horizontal barrier or "floor" to the cutoff structure to prevent vertical seepage (Figure 5.2c). The construction of horizontal barriers is relatively rare, but has been carried out using jet grouting, mix-in-place, permeation grouting, and artificial ground-freezing techniques.

If a complete physical cutoff is achieved, some groundwater will be trapped inside the working area. This will need to be removed to allow work to proceed, either by sump pumping during excavation or by pumping from wells or wellpoints before excavation.

One of the attractive characteristics of the exclusion technique is that it allows work to be carried out below groundwater level with minimal effects on groundwater levels outside the site. This means that most of the potential environmental effects of dewatering (see Chapter 15) are avoided. In particular, in urban areas, exclusion methods are often used in preference to dewatering methods to reduce the risk of settlement damage resulting from lowering of groundwater levels. However, when considering using exclusion techniques to avoid groundwater lowering in areas outside the site, it is essential to remember that almost all cutoff walls will leak to some extent. Leakage may particularly occur through any joints (between columns, panels, piles, etc.) resulting from the method of formation.

Leakage of groundwater through cutoffs into the excavation or working area can cause a number of problems:

1. During construction, the seepages may interfere with site operations, necessitating the use of sump pump or surface water control methods to keep the working area free of water.
2. The leakage into the excavation may be significant enough to locally lower groundwater levels outside the site, creating the risk of settlement or other side effects.

Table 5.1 Principal methods for groundwater control by exclusion

Method	*Typical applications*	*Notes*
Displacement barriers		
Steel sheet-piling	Open excavations in most soils, but obstructions such as boulders or timber balks may impede installation	May be installed to form permanent cutoff, or used as temporary cutoff with piles removed at the end of construction. Rapid installation. Can support the sides of the excavation with suitable propping. Seal may not be perfect, especially if obstructions are present. Vibration and noise of driving may be unacceptable on some sites, but "silent" methods are available in which piles are pressed into the ground by hydraulic jacks. Relatively cheap
Vibrated beam wall	Open excavations in silts and sands. Will not support the soil	Permanent. A vibrating H-beam is driven into the ground and then removed. As it is removed, grout is injected through nozzles at the toe of the pile to form a thin, low-permeability membrane. Rapid installation. Relatively cheap, but costs increase greatly with depth
Excavated barriers		
Slurry trench wall using cement-bentonite or soil-bentonite	Open excavations in silts, sands, and gravels up to a permeability of about 5×10^{-3} m/s	Permanent. The slurry trench forms a low-permeability curtain wall around the excavation. Quickly installed and relatively cheap, but cost increases rapidly with depth
Concrete diaphragm walls	Side walls of excavations and shafts in most soils and weak rocks, but the presence of boulders may cause problems	Permanent. Support the sides of the excavation and often form the sidewalls of the finished construction. Can be keyed into rock. Minimum noise and vibration. High cost may make the method uneconomical unless walls can be incorporated into the permanent structure
Bored pile walls (secant and contiguous)	As concrete diaphragm walls, but penetration through boulders may be costly and difficult	As concrete diaphragm walls, but more likely to be economic for temporary works use. Sealing between contiguous piles can be difficult, and additional grouting or sealing of joints may be necessary
Injection barriers		
Permeation and rock grouting using cement-based grouts	Tunnels and shafts in gravels and coarse sands, and fissured rocks	Permanent. The grout fills the pore spaces, preventing the flow of water through the soil. Equipment is simple and can be used in confined spaces. A comparatively thick zone needs to be treated to ensure a continuous barrier is formed. Multiple stages of treatment may be needed

Permeation and rock grouting using chemical and solution grouts	Tunnels and shafts in medium sands (chemical grouts), fine sands and silts (resin grouts), and fissured rocks	As cement-based grouting, but materials (chemicals and resin) can be expensive. Silty soils are difficult and treatment may be incomplete, particularly if more permeable laminations or lenses are present
Jet grouting	Open excavations in most soils and very weak rocks	Permanent. Typically forms a series of overlapping columns of soil/grout mixture. Inclined holes possible. Can be messy and create large volumes of slurry. Risk of ground heave if not carried out with care. Relatively expensive
Mix-in-place walls	Open excavations in most soils and very weak rocks	Permanent. Overlapping columns or panels formed by *in situ* mixing of soil and injected grout. Columns formed using auger-based equipment, panels formed using cutter soil mixing equipment. Produces little spoil. Less flexible than jet grouting. Relatively expensive
Thermal barriers		
Artificial ground-freezing using brine or liquid nitrogen	Tunnels and shafts. May not work if groundwater flow velocities are excessive (>2 m/day for brine or >20 m/day for liquid nitrogen)	Temporary. A "wall" of frozen ground (a freezewall) is formed, which can support the side of the shaft as well as excluding groundwater. Liquid nitrogen is expensive but quick; brine is cheaper but slower. Liquid nitrogen is preferred if groundwater velocities are relatively high. Plant costs are relatively high
Tunneling methods		
Compressed air	Confined chambers such as tunnels, sealed shafts, and caissons	Temporary. Increased air pressure (up to 3.5 bars) raises pore water pressure in the soil around the chamber, reducing the hydraulic gradient, and limiting groundwater inflow. Potential health hazards to workers. Air losses may be significant in high-permeability soils. High running and setup costs
EPB TBM	Tunnels in most soils and weak rocks	Temporary. The TBM excavates for the tunnel, and supports the soil and excludes groundwater by maintaining a balancing fluid pressure in the plenum chamber immediately behind the cutting head. The fluid is a mixture of soil cuttings, groundwater, and conditioning agents (such as polymer or bentonite muds). TBMs need to be carefully selected to deal with given ground conditions; setup and running costs may be high

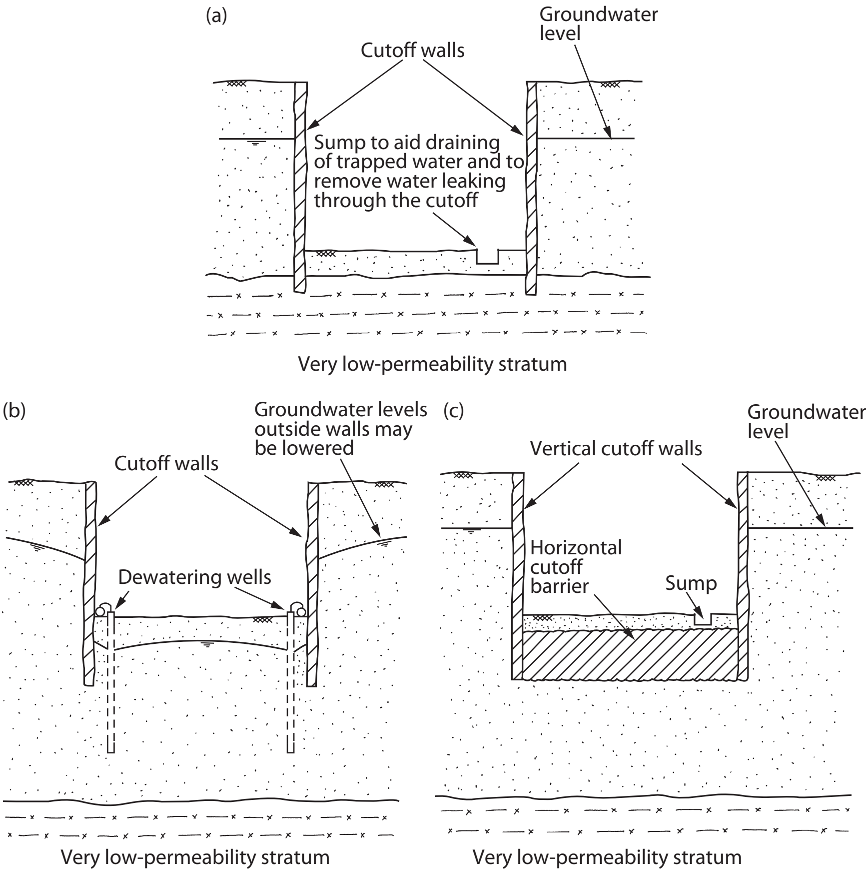


Figure 5.2 Groundwater control by exclusion using physical cutoffs. (a) Cutoff walls penetrate into a very low-permeability stratum. (b) Cutoff walls used in combination with dewatering methods. (c) Cutoff walls used with a horizontal barrier to seal the base.

3. If the cutoffs form part of a permanent structure (such as the walls of a deep basement), even very small seepages will be unsightly in the long term and may cause problems with any architectural finishes applied to the walls.

In many cases, the significant seepages that give rise to problems (1) and (2) can be dealt with by grouting or other treatment. On the other hand, it can be very difficult to prevent or to seal the small seepages from problem (3). David Greenwood (1994) said:

"Water penetration is very difficult to oppose. It is comparatively easy to reduce torrents to trickles, but to eliminate trickles is difficult. If it is essential to have a completely dry or leak-proof structure, costs rise steeply."

This is an important point to consider if cutoff walls are to be incorporated in the permanent works.

5.5 DEWATERING METHODS

Dewatering methods control groundwater by pumping, affecting a local lowering of groundwater levels (Figure 5.3). The aim of this approach is to lower groundwater levels to a short distance (~0.5 m) below the deepest excavation formation level.

The two guiding principles for securing the stability of excavations and especially of slopes, by dewatering or groundwater lowering are:

1. Do not hold back the groundwater. This may cause a buildup of pore water pressures that will eventually cause catastrophic movement of soil and groundwater.
2. Ensure that "fines" are not continuously transported because this will result in erosion and consequent instability. Apply a suitable filter blanket to avoid any buildup of pore water pressures and prevent transportation of fines. As a general guide, if the permeability is approximately less than 1×10^{-7} m/s, migration of fines ceases (this is a helpful guide when water testing after grout treatment, for example, before

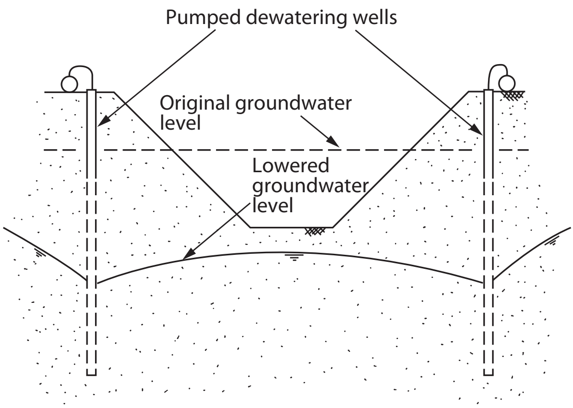


Figure 5.3 Groundwater control by pumping.

shaft sinking or the like, to assess whether or not further grouting is desirable).

These objectives can be achieved by lowering groundwater levels sufficiently to avoid groundwater seeping into the excavation. This is the basis of the so-called "predrainage" methods such as wellpointing (Chapter 9), deep wells (Chapter 10), and ejectors (Section 11.2) where groundwater levels are lowered *in advance* of excavation. Wells are typically installed outside the excavation (see Figure 5.3); this draws water away from the excavation, avoiding troublesome seepages and resulting instability.

The alternative approach is to allow groundwater to enter the excavation and then deal with it by sump pumping (Chapter 8); this is less than ideal because it draws water toward the excavation and may promote instability. Nevertheless, this can be acceptable, provided that fines are not removed from the slopes and base of the excavation. This philosophy requires pumping and disposal of the seepage water collected and the creation of a filter blanket and collector drainage system. The filter blanket may be a sand or gravel layer of adequate thickness (~0.5 m) and appropriate grading, or a suitable geotextile membrane held in place by some weighting materials (for example, cobbles and boulders). Measures necessary to avoid the instability of excavations under these conditions are described in Section 4.7.

There are several techniques or methods available for controlling groundwater flow for a construction project. The selection of a technique or techniques appropriate to a particular project at a particular site or country will depend on many factors. However, the lithology and permeability(ies) of the soils will always be of paramount importance. Other factors to be considered are:

- Extent of the area of construction requiring dewatering
- Depth of the deepest formation level below existing ground level and the amount of lowering required
- Proximity of existing structures, the nature of their foundations, and the soil strata beneath them

The various dewatering techniques are tabulated in Table 5.2, which is expanded from information in Preene et al. (2000).

There are projects in which a single method is insufficient and a combination of methods is appropriate. Where the excavation is to penetrate a succession of soils of widely varying lithology, this problem is more likely to arise.

In Table 5.2, the column "Typical applications" reveals that only a few methods are suitable for use in all types of soils. The ranges of soils that are suitable for treatment using the various dewatering methods are shown (Figure 5.4) in the traditional form of particle size distribution curves.

Table 5.2 Principal methods for groundwater control by pumping

Method	*Typical applications*	*Notes*
Drainage pipes or ditches (e.g., French drains)	Control of surface water runoff and shallow groundwater (including overbleed and perched water)	Simple methods of diverting or removing surface water from the working area. May obstruct construction traffic, and will not control groundwater at depth. Unlikely to be effective in reducing pore water pressures in fine-grained soils
Sump pumping	Shallow excavations in clean, coarse soils, for control of groundwater and surface water	Cheap and simple. May not give sufficient drawdown to prevent seepage from emerging on the cut face of a slope, possibly leading to loss of fines and instability. May generate silt or sediment laden discharge water, causing environmental problems
Wellpoints	Generally shallow, open excavations in sandy gravels down to fine sands and possibly silty sands. Deeper excavations (requiring >5–6 m drawdown) will require multiple stages of wellpoints to be installed	Relatively cheap and flexible. Quick and easy to install in sands. Suitable for progressive trench excavations. Difficult to install in ground containing cobbles or boulders. Maximum drawdown is ~5–6 m for a single stage in sandy gravels and fine sands, but may only be ~4 m in silty sands
Horizontal wellpoints (machine laid)	Generally shallow trench or pipeline excavations or large open excavations in sands and possibly silty sands	Horizontal wellpoints, laid by specialist trenching machines are suitable for long runs of trench excavations outside urban areas, where very rapid installation is possible
Deep wells with electric submersible pumps	Deep excavations in sandy gravels to fine sands and water-bearing fissured rocks	No limit on drawdown in appropriate soil conditions. Installation costs of wells are significant, but fewer wells may be required compared with most other methods. Close control can be exercised over well screen and filter
Deep wells with electric submersible pumps and vacuum	Deep excavations in silty fine sands, where drainage from the soil into the well may be slow	No limit on drawdown in appropriate soil conditions. More complex and expensive than ordinary deep wells because of the separate vacuum system. Number of wells may be dictated by the requirement of achieving an adequate drawdown between wells, rather than the flow rate, and an ejector system may be more economical

(*continued*)

Table 5.2 (*Continued*) Principal methods for groundwater control by pumping

Method	*Typical applications*	*Notes*
Shallow bored wells with suction pumps	Shallow excavations in sandy gravels to silty fine sands and water-bearing fissured rocks	Particularly suitable for coarse, high-permeability materials in which flow rates are likely to be high. Useful where correct filtering is important because closer control can be exercised over the well filter than with wellpoints. Drawdowns limited to ~4–7 m depending on soil conditions
Ejectors	Excavations in silty fine sands, silts, or laminated or fissured clays in which pore water pressure control is required	In practice, drawdowns are generally limited to 20–50 m depending on the equipment. Low energy efficiency, but this is not a problem if flow rates are low. In sealed wells, a vacuum is applied to the soil, promoting drainage. Inclined holes possible
Passive relief wells and sand drains	Relief of pore water pressure in confined aquifers or sand lenses below the floor of the excavation to ensure basal stability	Cheap and simple. Create a vertical flowpath for water into the excavation; water must then be directed to a sump and pumped away
Collector wells	Tunnels or deep excavations in relatively permeable soils such as sands and gravels, where surface access does not allow the installation of a large number of wells	Each collector well is expensive to install, but relatively few wells may produce large flow rates and be able to dewater large areas
Siphon drains	Long-term slope drainage and landslide stabilization in low-permeability soils	Siphon drains can allow passive drainage of slopes, without the need for pumping
Artificial recharge	Soils of high to moderate permeability and fissured rocks, in which lowering of groundwater is to be controlled so that environmental effects can be mitigated	Recharge systems are complex to operate and maintain. Recharge wells often suffer from clogging due to water chemistry effects and may require periodic backflushing and cleaning
Electroosmosis	Very low-permeability soils, e.g., clays, silts, and some peats	Only generally used for pore water pressure control or ground improvement when considered as an alternative to ground freezing. Installation and running costs are comparatively high

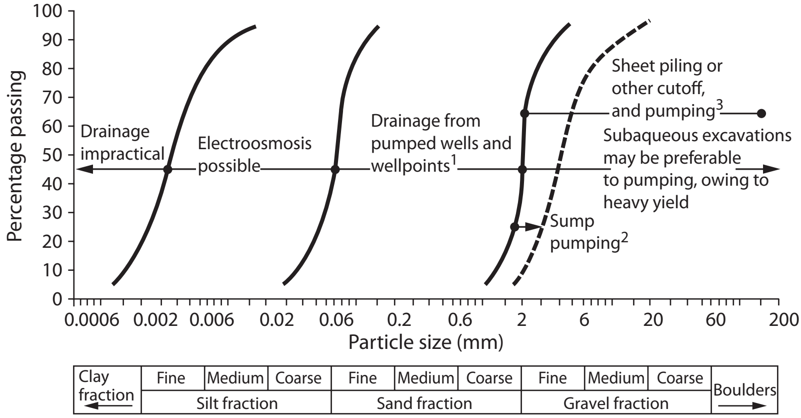


Figure 5.4 Tentative ranges for groundwater lowering methods. (1) Wellpoints in fine sands require good vacuum; (2) zone may be extended in finer soils by using large sumps with gravel filters; (3) to reduce the high water pressure on sheet piling, it may be preferable to control pumping as excavation proceeds and to install the support system as the water level is lowered. (From Somerville, S. H., Control of groundwater for temporary works. *CIRIA Report 113*, Construction Industry Research and Information Association, London, 1986. Reproduced by kind permission of CIRIA: www.ciria.org.)

These curves are taken from CIRIA Report 113 (Somerville 1986) and are based on the earlier work of Glossop and Skempton (1945) and others. Similarly, the ranges of soils suitable for treatment using the various exclusion methods are shown in Figure 5.5. These are tentative economic and physical limits. The emphasis is on the word *tentative*.

An interesting and useful variation of Figure 5.4 was presented by Roberts and Preene (1994a). Figure 5.6 is taken from CIRIA Report C515 (Preene et al. 2000) and is based on the Roberts and Preene (1994a) original *Range of Application of Construction Dewatering Systems* article, but has been modified in the light of the joint experiences of the authors (Cashman 1994) and others. The practical achievements at the lower end of the permeability range are constrained by the physical limitations of the applied vacuum. At the upper end of the range of permeability values, the cost of pumping the ensuing large volumes of water is the constraining factor. The greater the permeability, the greater the rate of pumping necessary to achieve the required lowering: the energy costs increase at an alarming rate.

The upper economic limit for the use of a deep well or wellpoint system is of the order of 5×10^{-3} m/s. The pumping costs of dealing with soils of greater permeability are generally uneconomic—except in the almost unheard of situation in which fuel or electrical power are provided at no

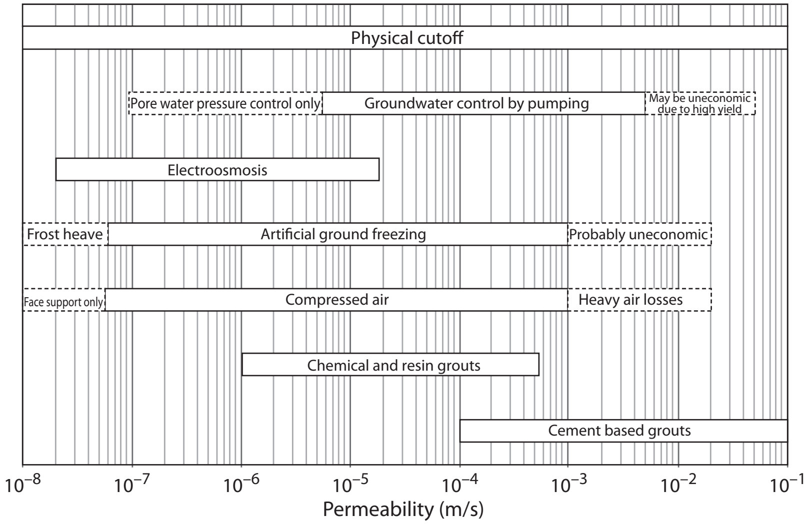


Figure 5.5 Tentative economic ranges for exclusion methods in soils. (Amended from Doran, S. R., Hartwell, D. J., Kofoed, N., and Warren, S. Storebælt Railway Tunnel—Denmark: design of cross passage ground treatment. *Proceedings of the 11th European Conference on Soil Mechanics and Foundation Engineering*, Copenhagen, Denmark, 1995; and Preene, M., Roberts, T. O. L., Powrie, W., and Dyer, M. R., Groundwater control—design and practice. *CIRIA Report C515*. Construction Industry Research and Information Association, London, 2000.)

cost. In such high-permeability soils, exclusion methods may offer a more cost-effective expedient (see Section 5.7).

5.5.1 Pore water pressure control systems in fine-grained soils

When dewatering methods are applied to high- and medium-permeability soils, local lowering of groundwater levels occurs by gravity drainage in response to pumping. The period of pumping necessary to lower the water level is quite short as the pore water is rapidly replaced by air. In fine-grained soils of low and very low-permeability, gravity drainage of the pore water is resisted by capillary tension. Such soils drain poorly and slowly by gravity drainage.

Because these soils do not drain easily, any excavation made below the groundwater level will encounter only minor seepages, and is unlikely to flood rapidly. Yet, even the small seepages encountered (perhaps less than 1 L/s even for a large excavation) can have a dramatic destabilizing effect. Side slopes may collapse or slump inwards and the base may become unstable

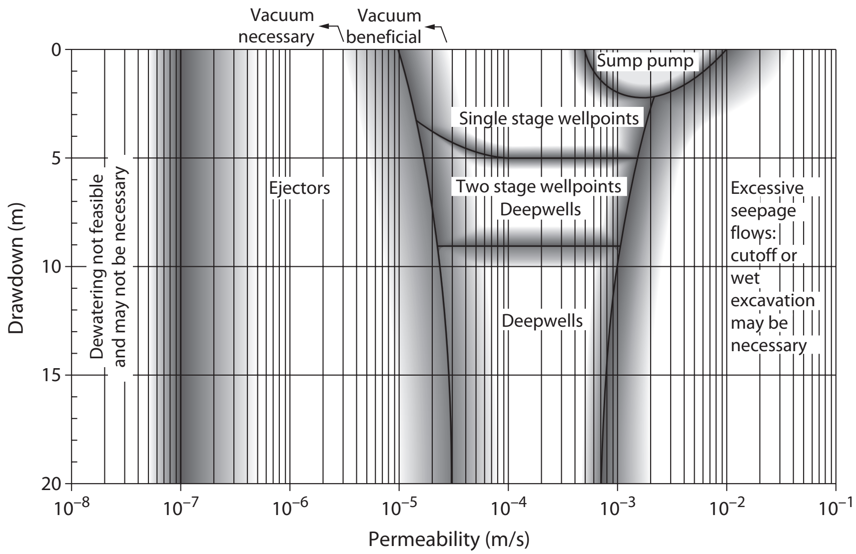


Figure 5.6 Range of application of pumped well groundwater control techniques. (Adapted from Roberts, T. O. L., and Preene, M. *Groundwater Problems in Urban Areas*, Wilkinson, W. B., ed. Thomas Telford, London, 415–423, 1994; and modified after Cashman, P. M., *Groundwater Problems in Urban Areas*, Wilkinson, W. B., ed., Thomas Telford, London, 446–450, 1994. From Preene, M. et al., Groundwater control-design and practice, Construction Industry Research and Information Association, *CIRIA Report C515*, London, 2000.Reproduced by kind permission of CIRIA: www.ciria.org.)

or "quick." On site, people are often surprised that such small flow rates can be a problem. The theory of effective stress explains the mechanism of instability (see Chapter 4). The seepages imply the presence of high positive pore water pressures around and beneath the excavation. This implies low levels of effective stress, and hence, low soil strength—instability is the natural result.

The solution to this problem is to abstract groundwater and so lower the pore water pressures around and beneath the excavation. This will maintain effective stresses at acceptable levels and prevent instability. The aim is not to totally drain the pore water from the soils—in any event, this would be very difficult as capillary forces mean that fine-grained soils can remain saturated even at negative pore water pressures (see Figure 3.8). Because the soil is not being literally "dewatered," pumped well systems in fine-grained soils are more correctly referred to as pore water pressure control systems, rather than dewatering systems. The application of pumped well systems in low-permeability soils is discussed further by Preene and Powrie (1994).

Glossop and Skempton (1945) and Terzaghi et al. (1996) indicated that gravity drainage will prevail in soils of permeability greater than approximately 5×10^{-5} m/s. Where wells are installed as part of pore water pressure control systems in soils of lesser permeability, the well yields will be very low. This can make the continued operation of conventional wellpoint or deep well systems difficult because the pumps are prone to overheating at low flow rates. However, if the top of the wells are sealed, a partial vacuum can be applied to assist drainage. The increase in yield of a well due to the application of a vacuum is likely to be of the order of 10%, sometimes up to 15%.

A sealed wellpoint system (see Section 9.6) operated by vacuum tank pump (see Section 13.2) may be effective in soils of permeability down to about 1×10^{-6} m/s. If a vacuum is applied to sealed deep wells (see Section 10.9), the lower limit of effectiveness of deep wells may be extended to about 1×10^{-5} m/s. Ejectors (see Section 11.2) installed in sealed wells will automatically generate a vacuum in the well when yields are low. Ejector systems can be effective in soils of permeability as low as 1×10^{-7} m/s.

Again, the permeability ranges quoted above are tentative. In fine-grained soils, structure and fabric has a great influence on the performance of vacuum well systems. If the soil structure consists of thin alternating layers or laminations of coarser and finer soils, the drainage will be more rapid. This is because the layers of coarse material will more rapidly drain the adjacent layers of finer-grained soil. There have also been cases in which pore water pressure control systems have been effective in clays of very low-permeability—the success of the method was attributed to the presence of a permeable fissure network in the clay (see, for example, Roberts et al. 2007).

5.5.2 Some deep well and ejector projects deeper than 20 m

The 20-m depth limit on Figure 5.6 is not restrictive, but is merely a convenience for the presentation of the diagram. However, there are an insufficient number of case histories reported to have confidence in extending the data from Figure 5.6 below the 20-m depth limit, but four known case histories are reported below to substantiate the view that, with care, it is economically feasible to use wells and ejectors to depths of the order of 40 m or more.

At Dungeness A Nuclear Power Station, sited on the south coast of England, sixty deep wells were pumped for more than 2 years. The soil conditions at the site were:

1. Gravel with cobbles, +5·5 meters above Ordnance Datum (mOD) to –1·5 mOD, average permeability 3×10^{-3} m/s
2. Original groundwater level from +2·4 mOD to 0·9 mOD

3. Gravelly sand, –1·5 mOD to –9·1 mOD, average permeability 8×10^{-4} m/s
4. Sand, –9·1 mOD to –33·5 mOD, permeability $1{\cdot}5 \times 10^{-4}$ m/s decreasing with depth to 5×10^{-5} m/s

The construction excavations were encircled by a continuous girdle of interlocking sheet piles driven to –9·1 mOD level so as to exclude recharge from the overlying high-permeability soils. The lowering for the excavations for the Turbine Hall and the Reactors was achieved by pumping twenty-nine wells 20 m deep. The lowering for the Cooling Water Pumphouse and the Syphon Recovery Chamber was achieved by pumping nineteen and twelve wells, respectively, each 41 m deep.

Before the construction of the East Twin Dry dock in Northern Ireland, site investigation borings had revealed a confined aquifer of Triassic sandstone beneath alluvial and glacial deposits (both mainly clays). The depth to the upper surface of Triassic sandstone varied between 34 m below ground level and 21 m (which was 1 m below the formation level of the entrance structure). There was a high piezometric level confined in the sandstone—to about 3 m above the original ground level. Twenty deep wells were installed to depths between 33.5 and 43 m below ground level using reverse circulation rotary drilling methods. The wells were operated for more than 2 years.

At the Mufulira mine no. 3 dump, in northern Zambia, approximately two hundred twin pipe ejectors were installed to reduce the moisture content and thereby stabilize slime lagoon deposits, which were of porridge-like consistency and had broken through the roof of the main adit causing disastrous loss of life. The Ministry of Mines in Zambia required that the slime deposits over the cave-in be stabilized before the mine could be permitted to reopen. After 9 months of pumping of the ejector installation, the phreatic surface was drawn down about 10 m. Eventually, the phreatic surface was drawn down some 30 m and resulted in an acceptable increase in the shear strength of the slime deposits over the cave-in. The pumping lift was in the range of 20 m to more than 40 m.

At the Benutan Dam Site in Brunei, single pipe ejectors were installed to stabilize alluvium comprising heterogeneous loose silty fine sands, very soft clays, thin layers of peat, and buried timbers (Cole et al. 1994). Brunei is in an earthquake zone of moderate seismic activity, so it was judged that there was a risk of liquefaction of these loose alluvial foundation soils beneath the proposed dam unless they were stabilized or replaced with other more stable materials. Similar with the Mufulira site, an ejector system was installed and operated to reduce the pore water pressures of the loose soft soils and thereby increase their stability. The depths to which the ejectors were installed ranged from about 10 to 38.5 m. The ejector pumping was continuous on the deepest line for about 2 years. The dam height was 20 m above the valley floor level.

5.6 GROUNDWATER CONTROL FOR TUNNELS AND SHAFTS

There is some merit in considering groundwater control for tunnels and shafts separately from methods used for surface excavations (although cut-and-cover tunnels are effectively surface excavations and can be dewatered accordingly). Even though the great majority of tunnels are constructed below groundwater level, traditionally, most tunnels through water-bearing ground are constructed without dewatering by pumping from wells.

Tunnel projects also commonly involve the construction of shafts between which the tunnel is driven. Other shafts may be needed for access as part of the completed scheme. Dewatering pumping is often used for shaft construction, although many shafts are constructed as flooded or "wet" caissons. This method involves the shaft lining being constructed at ground level and sunk (by jacking or kentledge) into the ground while material is excavated by grabbing from within the flooded caisson. This avoids the need to lower groundwater levels, but carries its own set of risks. It can be difficult to control excavation levels when grabbing through considerable depths of water, and problems have occurred because of overexcavation, or when boulders are present.

5.6.1 Tunneling-specific methods

A tunnel is a challenging working environment at the best of times. Any groundwater ingress is only going to add to the difficulties. At the tunnel face, you are in a confined space, deep below ground, and access is often difficult if it is needed to deploy additional pumps or ground treatment equipment. It is essential that groundwater ingress is prevented or managed in a safe and appropriate manner.

Because many tunneling methods construct a relatively watertight lining immediately behind the working face, for tunnels in soils and soft rocks, typically only a small area of tunnel face is exposed to water-bearing ground. In many cases, even in relatively permeable soils, the rates of groundwater ingress may not be so large and can be managed by simply pumping away along the tunnel to the surface. However, groundwater control requirements in tunnels are often not merely about preventing the works from being "flooded out." Modern tunneling contracts are generally planned on the basis of rapid and efficient tunnel progress, which requires stable and controllable ground behavior at the face, and tunnel spoil (colloquially called "muck") that is easy to handle. The presence of uncontrolled groundwater can seriously slow down tunnel progress. Water seeping through the face could cause localized instability and collapse (especially in silts and sands), and the presence of groundwater in the tunnel will make the muck wet and difficult to handle. Effective control of groundwater can have huge benefits to the efficiency of tunneling operations and pay back the cost of groundwater control measures several times over.

The confined environment associated with the working space around the tunnel face has allowed tunneling-specific methods to be developed whereby groundwater is excluded by maintaining the tunnel face at a fluid pressure more or less equal to the groundwater pressure.

The traditional way of balancing groundwater pressure is by compressed air work (Figure 5.7a). This method, in use since the nineteenth century, uses pressurized air in the tunnel face and working area (for

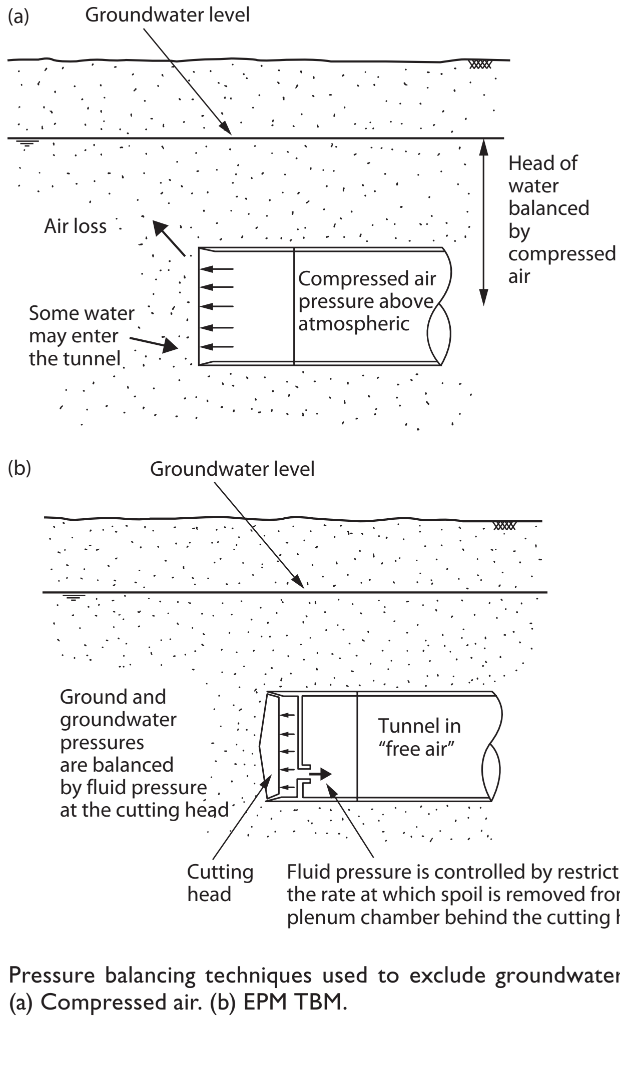


Figure 5.7 Pressure balancing techniques used to exclude groundwater from tunnels. (a) Compressed air. (b) EPM TBM.

some distance from the face), requiring the miners to work in air pressures above atmospheric, with concomitant health risks. One bar of air pressure will balance around 10 m head of groundwater. When tunneling through a typical unconfined aquifer, the groundwater pressure will vary across the face, being lowest at the crown and highest at the invert of the tunnel (for a 3-m diameter tunnel, the difference in groundwater pressure between the crown and invert will be around 0.3 m). It is never possible to get perfect air pressure in the tunnel to balance water pressures across the whole face. For example, if the air pressure is set to balance the water pressure at the median height of the tunnel, some air loss may occur in the crown, and water may enter the tunnel above the invert.

Working under compressed air conditions can be difficult and expensive. Physiological effects on workers exposed to compressed air mean that the method can only be used up to 3.5 bar of pressure above atmospheric (equivalent to 35 m below water level), and only then with very careful medical controls on working arrangements. Most compressed air work is carried out at pressures less than 0.75 bar above atmospheric (known as low-pressure compressed air); costs increase considerably above 0.75 bar (high-pressure compressed air) due to the additional medical constraints. Compressed air work can also be used for shaft construction, when an air deck and airlocks are used to seal the top of the shaft. Further information on compressed air work is given in Megaw and Bartlett (1981).

An alternative method of balancing ground and groundwater pressures at the tunnel face was developed in the late twentieth century—the earth pressure balance (EPB) tunnel-boring machine (TBM). EPB machines are a form of full-face TBM that cuts the soil from the tunnel face, and forms a fluid or "earth paste" (consisting of soil cuttings, groundwater, and conditioning agents such as polymer or bentonite muds) in a plenum chamber behind the face (Figure 5.7b). By controlling the rate at which the fluid or paste is extruded from the plenum chamber as the TBM excavates and moves forward, the face can be supported by a balanced pressure. The TBM driver and miners work in a "free air" (i.e., atmospheric) environment behind the plenum chamber, avoiding the health risks of working in a compressed air environment.

5.6.2 Exclusion methods in tunneling

Ground treatment is often used to exclude groundwater from tunnels and shafts. Methods used include jet grouting or permeation grouting and artificial ground-freezing, carried out from within the tunnel or from the surface. Any residual seepage into the tunnel is dealt with by maintaining some sump pumping capacity at the tunnel face. More details on exclusion methods are given in Chapter 12.

Some of the exclusion methods, for example, jet grouting and artificial ground-freezing, have the advantage that, in addition to acting as a barrier to groundwater, they increase soil strength and can therefore act to provide some structural support to a shaft or tunnel. These methods are sometimes used as part of "recovery works" where there has been localized collapse or inundation of a tunnel. Clarke and Mackenzie (1994) and Brown (2004) describe two case studies in which artificial ground-freezing was used to allow completion of two water tunnels which encountered problems during construction. In both these cases, a vertical freezewall was formed around the shafts, and horizontal freezetubes were drilled around the tunnel to allow a cylinder of frozen ground to be formed around the damaged section of tunnel.

5.6.3 Pumping methods in tunneling

Cases do sometimes arise when groundwater control by pumping is used in tunnel construction. Hartwell (2001) describes several case histories. Applications of groundwater lowering for tunneling include:

1. For construction of shafts to launch or receive TBMs, and for construction of "soft eyes" in shaft linings to allow the TBM to enter or exit the shaft.
2. The entry or exit of TBMs into or out of shafts, portals, outfalls, and other structures.
3. For construction of tunnel enlargements, step plate junctions, cross passages, adits, or other connections where the tunnel lining has to be breached temporarily.
4. To lower groundwater levels to allow compressed air work at reduced pressures (ideally less than 1 bar), for example, to allow access to the working head of a TBM for maintenance purposes. The dewatering wells should be located with care to avoid compressed air escaping through the ground to the wells.
5. To reduce pore water pressures in fine-grained soils such as silts or very silty sands, to reduce the risk of the soils liquefying as a result of vibration from the machinery in the TBM.
6. To lower groundwater levels to below the invert (to effectively "dewater" the tunnel) to allow open-face tunneling methods to be used in otherwise unstable ground. This is sometimes necessary when a TBM, designed for relatively stable soils present over most of the tunnel length, has to traverse a short section of alluvial or glacial soils which may be present in a buried channel or other geological feature.
7. For recovery of damaged or inundated TBMs.
8. To control groundwater velocities to allow use of ground treatment methods (such as artificial ground-freezing or grouting) in problematic conditions.

Where dewatering is used for tunnel works, wells drilled from the surface can be used, provided that access is available above the tunnel, and that there are no intervening service pipes between ground level and the tunnel. If surface access is not available, it may be possible to drill small-diameter wells radially out from the tunnel or shaft (or both). This technique was used in the 1990s on the London Underground Jubilee Line Extension (Preene and Roberts 2002) and also on the Storebælt Railway Tunnel in Scandinavia (Doran et al. 1995). An example well arrangement is shown in Figure 5.8. Collector wells have also been used as part of tunnel dewatering schemes (see Section 14.7).

Drilling out through the existing tunnel lining is a challenging task, and runs the risk of destabilizing or inundating the tunnel; it is essential that such works are meticulously planned and executed, and supervised by experienced personnel. Drilling must normally be carried out through stuffing boxes and blow-out preventers secured and sealed to the tunnel lining. Care must be taken to avoid loosing ground into the tunnel during drilling and subsequent pumping—this is a particular risk in fine-grained uniformly graded soils such as silts and sands. Experience suggests that in such soils, if groundwater heads are in excess of around 10 m above the tunnel invert, successful well installation will be very difficult.

Occasionally, dewatering during tunnel construction may be achieved by drilling of horizontal directionally drilled wells along or immediately parallel to the tunnel route (see Section 11.4). These drains act as preferential groundwater flow pathways to feed water in a controlled manner to the tunnel face or to a reception shaft ahead of the tunnel, from where the water is pumped away. An example of such an application is given by Peter Cowsill Limited (2001).

On most urban tunnel projects, the use of groundwater pumping systems based on arrays of conventional vertical wells drilled from the surface is normally precluded by access restrictions and obstructions on the land above the tunnel route. Nevertheless, large-scale dewatering is occasionally carried out along extensive sections of tunnels. One interesting example is the Channel Tunnel Rail Link constructed in southern England in the late 1990s and early 2000s. Whitaker (2002) describes how a major dewatering exercise was carried out as part of the tunnel works, with the aim of significantly lowering groundwater pressures at the tunnel level to provide easier working conditions for the TBMs, and reducing the ground treatment requirements for cross passages. The scheme involved abstracting up to 700 L/s from the lower aquifer beneath London (Chalk and Thanet Sand) from 39 wells at 22 well site locations. This project faced some typical challenges for tunnel dewatering in urban areas. The project client did not own the land above the tunnel, so parcels of land close to the tunnel alignment had to be leased to provide dewatering well site locations. The existing sewer network did not have the capacity to handle the dewatering discharge

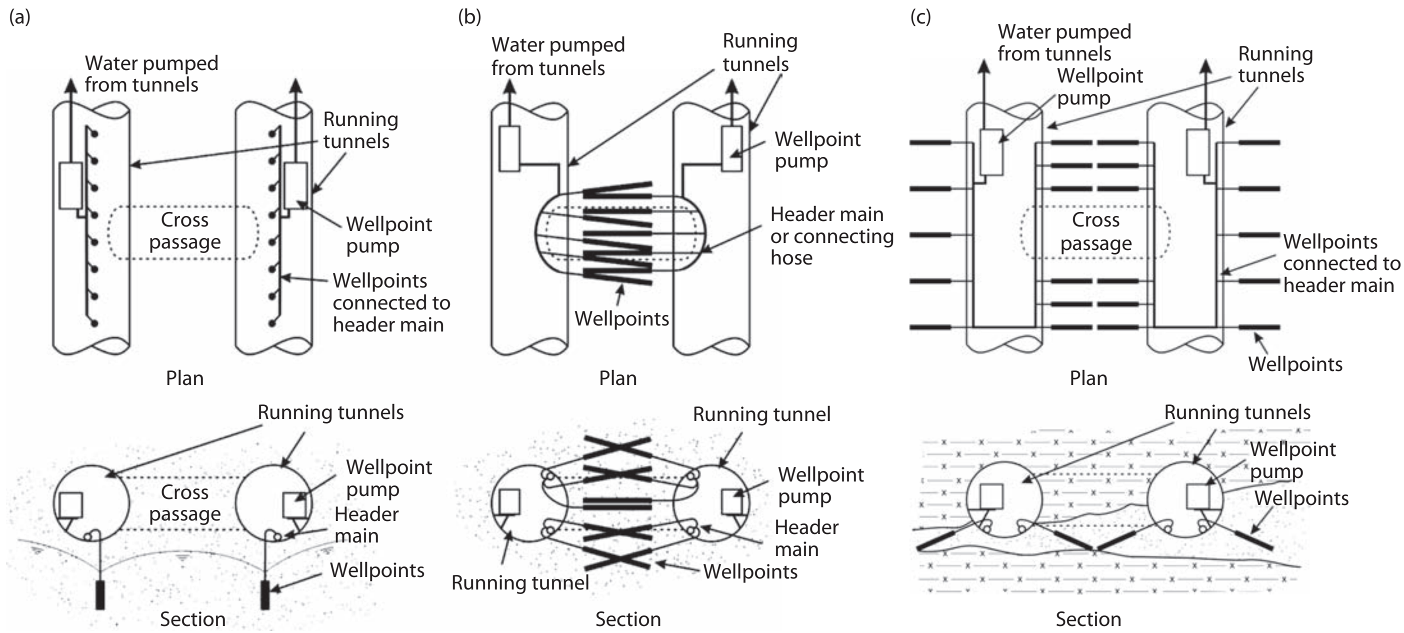


Figure 5.8 Array of wellpoints used for tunnel cross passage construction. (a) In uniform deposits of moderate permeability vertical or inclined wellpoints installed from invert of main tunnels may be able to reduce pore water pressures sufficiently to allow cross passage construction. (b) In uniform deposits of low permeability materials such as silts, wellpoints will need to be installed at close spacings in a fan around the space where the cross passage is to be constructed. (c) In deposits of variable permeability inclined wellpoints are installed from the main tunnels at a level and orientation where they are likely to intercept the permeable layers which require depressurization.

flow, so a dedicated 3.5-km-long collector pipeline (up to 600 mm in diameter) was laid beneath the streets in the area. A final interesting point about the project is that several of the dewatering wells were installed to a higher specification than is typical for dewatering wells, suitable for use as public water supply wells. These wells (and the water collector pipeline) were ultimately taken over by the local drinking water supply company, and are now used to supply raw water to the regional water supply system (Hamilton et al. 2008).

5.7 USE OF PUMPING AND EXCLUSION METHODS IN COMBINATION

Pumped groundwater control systems are frequently used in combination with exclusion methods. The methods might be combined for a number of reasons:

1. To reduce pumped flows in high-permeability soils
2. To reduce external groundwater effects
3. To dewater cofferdams formed by cutoff walls
4. To reduce loading on cutoff structures

5.7.1 Exclusion methods to reduce pumped flows in high-permeability soils

In high-permeability soils, where the pumped flow will be large, cutoff methods (sheet pile walls, slurry walls, grout barriers, etc.) can be used to reduce the potential inflows to an excavation. If the abstraction points (sumps or wells) are located within the cutoff walls, the pumped flow rate will be reduced to a lesser or greater degree depending on the effectiveness of the exclusion method.

5.7.2 Exclusion methods to reduce external groundwater effects

There are some hydrogeological and geotechnical conditions in which groundwater lowering is perfectly possible from a practical point of view, but in which there is a risk of detrimental effects on neighboring structures or natural features such as wetlands. Chapter 15 describes the full range of environmental effects potentially associated with groundwater lowering, but some of the more common effects include settlement damage of neighboring structures, the depletion of groundwater-dependent features such as rivers and wetlands, or the reduction in yield of nearby water supply wells.

Depending on the geological stratification at the site, exclusion methods such as cutoff walls or grout curtains can be used to fully or partially isolate the excavation from the wider groundwater regime. This will reduce the drawdown of groundwater levels external to the excavation and will generally reduce the corresponding external effects.

A typical dewatering problem associated with major infrastructure projects is one in which new infrastructure is constructed to replace or supplement an existing plant on the same site. When dewatering was carried out for the original infrastructure, the site will have effectively been "greenfield" with few constraints on dewatering. However, when the new infrastructure is constructed, the dewatering effect on the existing infrastructure must be considered carefully.

This is illustrated by the groundwater control works for Sizewell B Power Station in Suffolk, United Kingdom described by Howden and Crawley (1995) and Knight et al. (1996). In the late 1980s, the new "B Station" was being constructed immediately adjacent to the existing "A Station." The A Station had been constructed in the 1960s and its foundations had been dewatered using a relatively small number of wells pumping from the sandy Norwich Crag stratum. During planning for construction of the B Station, calculations showed that dewatering for the new foundations would induce unacceptable ground settlements beneath the A Station and also may have a detrimental effect on nearby marshland and farm irrigation systems.

The solution adopted was to enclose the entire perimeter of the B Station construction site with a 1259-m-long concrete diaphragm wall cutoff, extending to 56 m depth to key into the very low-permeability London Clay beneath the Norwich Crag deposit (Figure 5.9). A system of dewatering wells was operated within the area enclosed by the cutoff and lowered the water level to 16 m below sea level. Despite a localized defect in the wall (probably associated with a joint between panels in the diaphragm wall), which was sealed by grouting, groundwater levels outside the dewatered excavation remained very close to their natural level of around 1 m above sea level. Construction was completed with no significant dewatering-related external effects.

5.7.3 Groundwater pumping to dewater cofferdams

If the soil stratification allows the cutoff wall to form a complete cofferdam (e.g., if sheet piles are driven down to an impermeable layer) the excavation will be isolated from the surrounding groundwater regime. However, some groundwater will remain trapped inside the cofferdam, and will need to be pumped away. This is sometimes done by crude sump pumping as excavation proceeds. However, this may result in loss of fines or loosening of the soils that lie beneath the formation of the structure under construction. Sometimes, wells or wellpoints are installed inside the cofferdam to

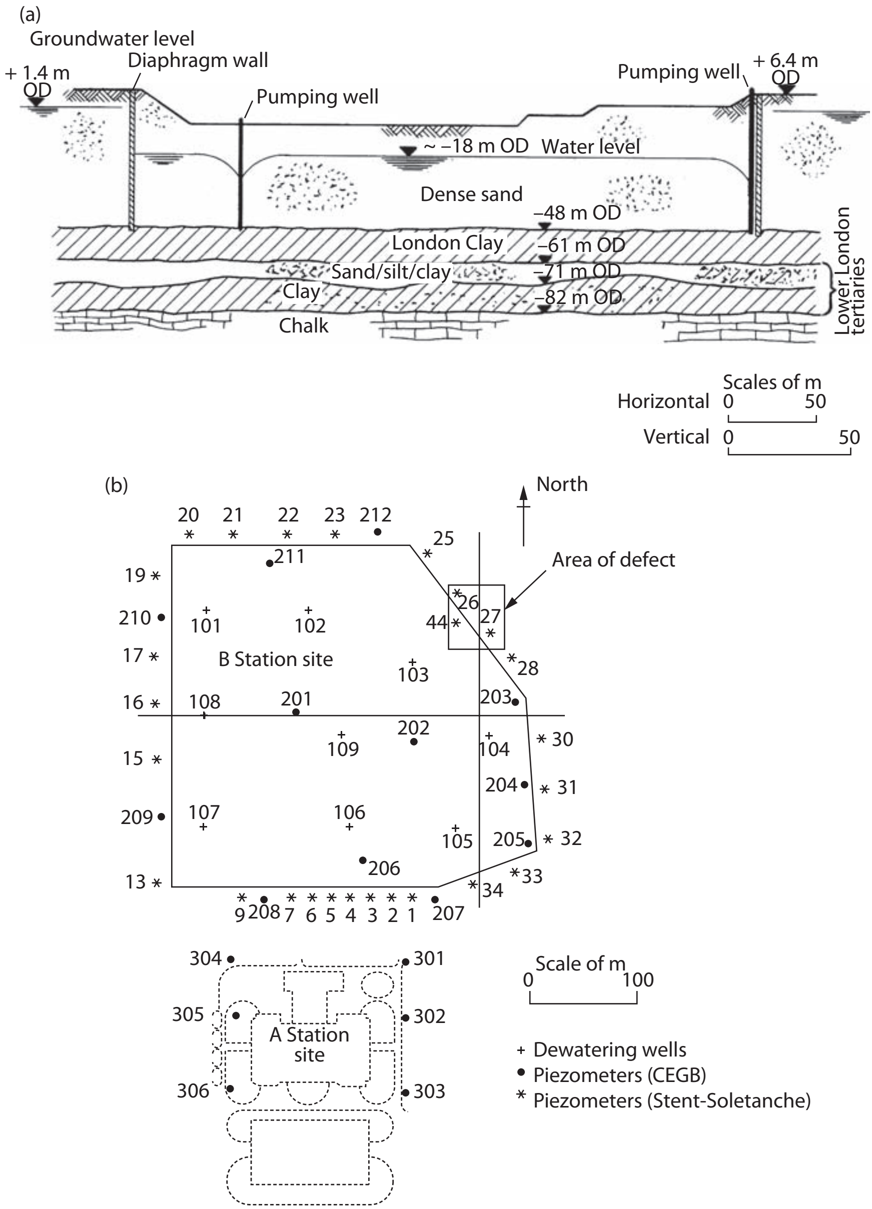


Figure 5.9 Use of diaphragm wall at Sizewell B Power Station. (a) Typical geological section showing diaphragm wall and dewatering wells. (b) Position of dewatering wells and monitoring piezometers. (From Howden, C., and Crawley, J. D., *Proceedings of the Institution of Civil Engineers, Civil Engineering, Sizewell B Power Station*, 108, 48–62, 1995; reproduced by permission of ICE Publishing.)

predrain water levels before excavation, thus reducing detrimental effects on the soil formation.

5.7.4 Groundwater lowering used to reduce loading on cutoff structures

If the sides of an excavation are temporarily supported by sheet pile or concrete diaphragm walls or bored pile walls, the stresses on the walls will increase partly from soil and partly from groundwater loading. In some circumstances, groundwater lowering can be used to reduce the groundwater loading on a wall, to minimize temporary propping requirements. This can be a highly effective expedient; fewer props in the construction area can allow work to proceed quicker and with less obstruction. One example in which this was done was for the construction of cut-and-cover tunnels on the Channel Tunnel Rail Link at Ashford, United Kingdom. Ejector wells were used to reduce pore water pressures outside a bored pile retaining wall in a very low-permeability fissured clay. The temporary depressurization achieved by the ejector wells decreased the load on the walls and allowed the number of levels of temporary props to be reduced (Roscoe and Twine 2001; Roberts et al. 2007).

However, a note of caution should be sounded. If pumping is interrupted (e.g., because of power or pump failure) the recovery of groundwater levels will increase loading on the walls and props, leading to overstressing, distortion and, in the worst case, collapse. Adequate standby facilities (perhaps arranged for automatic start-up) are essential. Alternatively, pressure relief holes could be formed through the wall above the formation level; if water levels rose, these would relieve the pressure behind the wall, albeit at the inconvenience of allowing the excavation to be flooded.

5.7.5 Example of optioneering of groundwater pumping and exclusion scheme

The interaction between different combinations of pumping and cutoff wall geometries on a given site can be complex, and will be influenced by the project-specific requirements for using the techniques together—for example, is a cutoff wall used to reduce pumping rates or to reduce external lowering of groundwater levels?

Figure 5.10 shows schematic sketches of some combinations of techniques which might be used when planning the dewatering of a circular shaft to be excavated into relatively competent but fissured (and therefore permeable) bedrock.

Figure 5.10a shows the simplest dewatering option, whereby external deep wells are used to lower groundwater levels and no cutoff wall is used. This option is likely to have a relatively low groundwater control

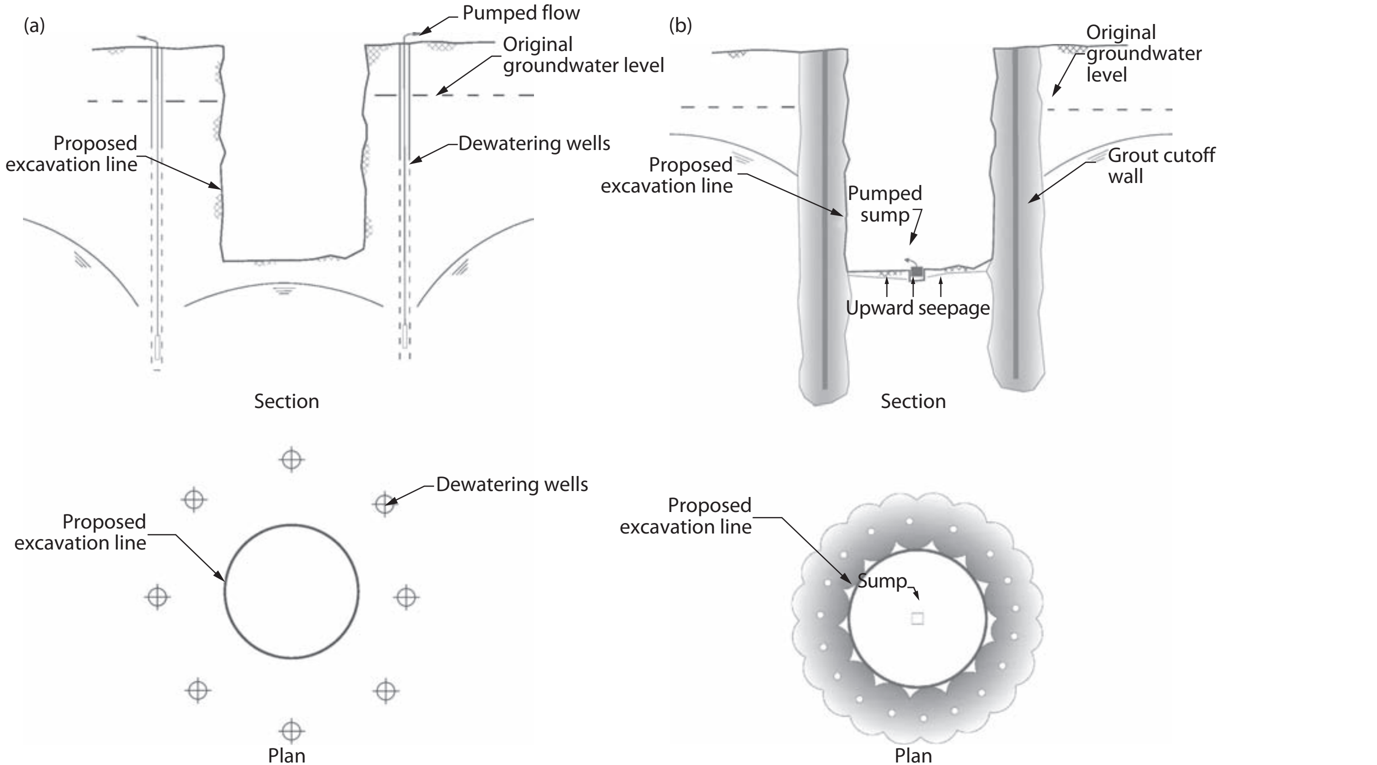


Figure 5.10 Optioneering for groundwater exclusion methods used in combination with pumping. (a) External pumping boreholes (no cutoff wall): (b) vertical grout curtain with sump pumping; (c) vertical grout curtain with relief wells; (d) vertical grout curtain with internal dewatering wells; and (e) and vertical grout curtain and grout basal seal.

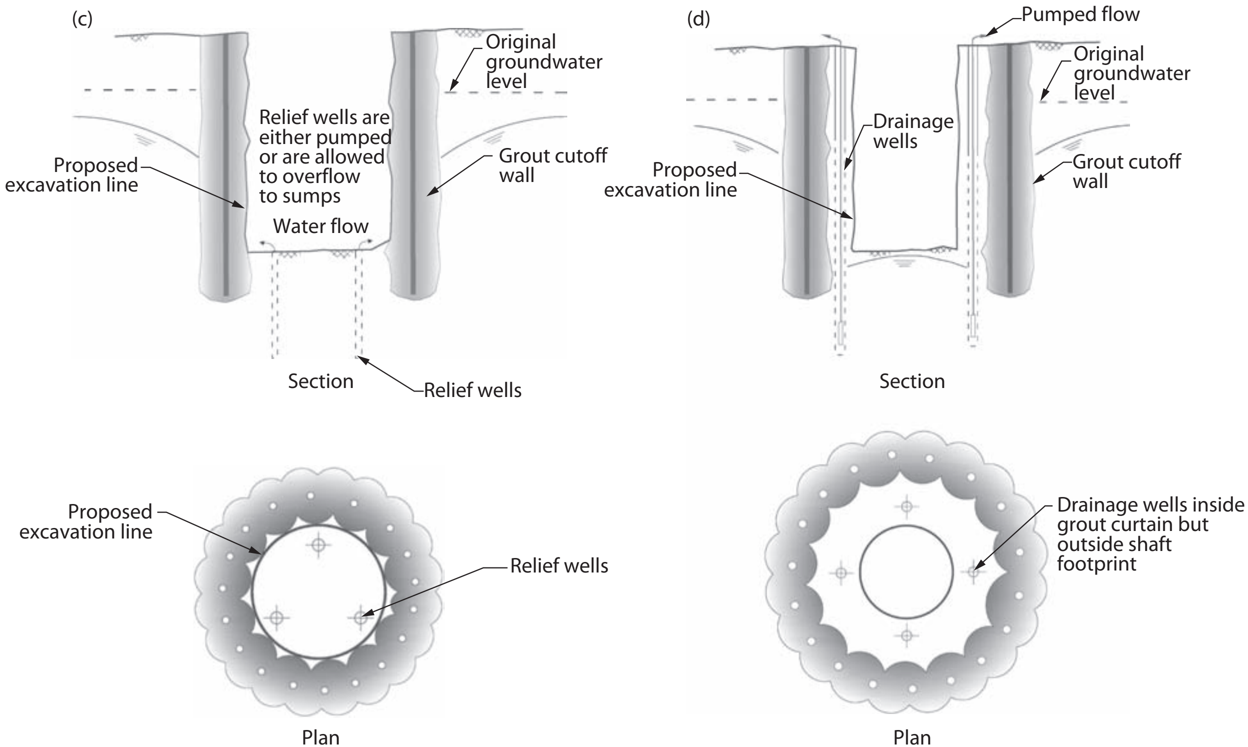


Figure 5.10 (Continued) Optioneering for groundwater exclusion methods used in combination with pumping. (a) External pumping boreholes (no cutoff wall): (b) vertical grout curtain with sump pumping; (c) vertical grout curtain with relief wells; (d) vertical grout curtain with internal dewatering wells; and (e) and vertical grout curtain and grout basal seal.

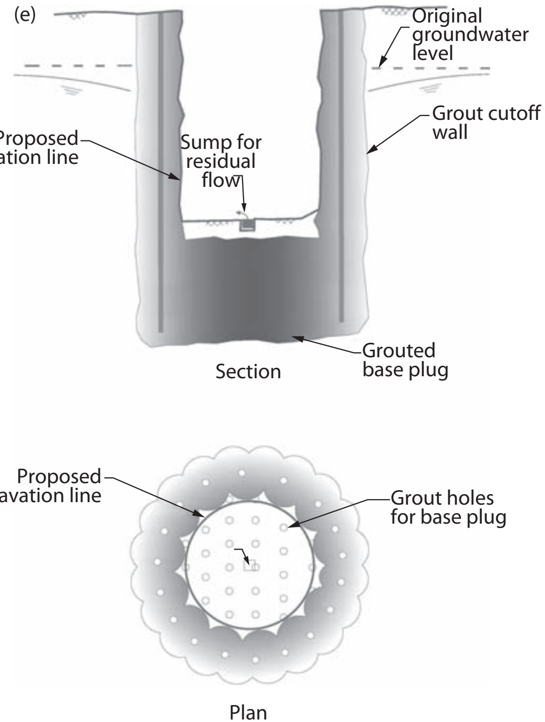


Figure 5.10 (Continued) Optioneering for groundwater exclusion methods used in combination with pumping. (a) External pumping boreholes (no cutoff wall): (b) vertical grout curtain with sump pumping; (c) vertical grout curtain with relief wells; (d) vertical grout curtain with internal dewatering wells; and (e) and vertical grout curtain and grout basal seal.

cost, but will potentially create an extensive zone of groundwater lowering around the shaft, which may affect neighboring structures or groundwater-dependent features.

If there is a concern about the effects of external drawdown, then a cutoff wall (in this case a grout barrier) could be installed around the perimeter of the shaft (Figure 5.10b). In this case, the base of the shaft is ungrouted, so water will still enter the base of the excavation, and will be collected in sumps and pumped away. The key challenges with this option are to manage the potentially sediment-laden water pumped from the sumps, and to ensure that upward hydraulic gradients are not so large that there is a risk of base instability (see Section 4.6). The addition of relief wells (Figure 5.10c) within the excavation will provide preferential vertical pathways for flow and will reduce the risk of base instability. If disposal of sediment-laden water from sumps is likely to be a problem, then deep wells could be installed within the area enclosed by the cutoff wall (Figure 5.10d). The wells are used to lower groundwater levels within the excavation, and provided they have suitable filters and are appropriately developed following drilling, should provide discharge water with much lower sediment loads than would be derived from sump pumping.

A vertical cutoff wall of the type shown in Figures 5.10b through 5.10d will reduce the lowering of groundwater levels outside the shaft; however, unless the cutoff wall either keys into a very low-permeability stratum or extends to a great depth, significant drawdowns may still occur around the shaft. If it is desired to reduce external drawdown to an absolute minimum, and there is no suitable very low-permeability stratum for the cutoff wall to seal into, then it may be appropriate to install a basal grout seal to link with the cutoff walls and close off the bottom of the excavation (Figure 5.10e). This is likely to be the most expensive groundwater control option but, if correctly implemented, will ensure that lowering of groundwater level outside the shaft is negligible.

These various options illustrate that pumping and exclusion methods can be used in a wide range of combinations. There is no single “best” combination of systems. The most appropriate choice for a given site and project will depend on ground conditions, economics, the sensitivity of surrounding areas to drawdown effects, as well as the technologies available at the site locality.

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Chapter 6

Site investigation for groundwater lowering

6.1 INTRODUCTION

Site investigation was defined by Clayton et al. (1995) as

> "The process by which geological, geotechnical, and other relevant information which might affect the construction or performance of a civil engineering or building project is acquired."

This chapter describes the information which needs to be gathered to allow the design of groundwater lowering systems. Good practice in the planning and execution of site investigations is outlined, together with brief details of methods used for boring, probing, testing, etc. The analysis of the data gathered during site investigation is described with the design methods in Chapter 7.

This chapter deals specifically with the site investigation requirements for groundwater control projects. The successful design of such projects is highly dependent on obtaining realistic estimates of the permeability of the various soil and rock strata present beneath the site. Methods of permeability testing, their range of applicability, and their relative merits and limitations are discussed in detail.

6.2 PURPOSE OF SITE INVESTIGATION

Site investigation (also known as geotechnical investigation) is the essential starting point, without which the design of any construction or geotechnical process cannot progress. Groundwater lowering is no exception to this rule. To quote the fictional detective Sherlock Holmes:

> "It is a capital mistake to theorize before one has data."
>
> (Sir Arthur Conan Doyle, *A Scandal in Bohemia*)

The best site investigations are deliberately planned and executed processes of discovery, carefully matched to the characteristics of the site, and to the work to be carried out. Sadly, in the past, many investigations have not met this challenging standard. The problems of poor or inadequate investigations were highlighted in two reports by the Institution of Civil Engineers: *Inadequate Site Investigation* (Institution of Civil Engineers 1991) and *Without Investigation Ground is a Hazard* (Site Investigation Steering Group 1993).

A particular problem for the designer of groundwater lowering works is that, in many cases, investigations are designed primarily to provide information for the design of the permanent works. The information needed for the design of temporary works (including groundwater lowering) is often neglected. This problem can arise when the persons designing the site investigation do not have appropriate expertise or experience. Alternatively, poor communication may result in them not being informed of the likely need for dewatering; therefore, they will not plan to gather the relevant information.

There is a wide and useful literature on site investigation, including Clayton et al. (1995), the *British Standard Code of Practice for Site Investigation* (BS 5930:1999, amended 2010), Eurocode 7 (BS EN 1997-1:2004; BS EN 1997-2:2007), and the Institution of Civil Engineers' reports cited previously. The reader should also consult these for background on the subject. The remainder of this chapter will concentrate on the particular site investigation needs for projects in which temporary works groundwater lowering schemes are to be used.

6.3 PLANNING OF SITE INVESTIGATIONS

All site investigations need to be planned and designed so that they provide the information needed by the various designers, estimators and construction managers. There is no such thing as a "standard" site investigation, purely because standard ground conditions have yet to be discovered, no matter how much we might wish for them!

According to the Institution of Civil Engineers (1991), the designers and planners of site investigations should attempt to answer the following questions:

1. What is known about the site?
2. What is not known about the site?
3. What needs to be known?

On all but the smallest investigations, these questions cannot be answered by one person and may require input from specialists in soil and rock mechanics, engineering geology, geophysics, archaeology, and hydrogeology. For groundwater lowering projects, advice from a dewatering specialist can also aid the planning of investigations.

The planning, design, and, ultimately, procurement, of site investigations is highly specialized. It is essential that this work is guided by a suitably qualified and experienced person, who should be associated with the site investigation from conception to completion. Recommendations for the qualifications and experience required to act as geotechnical specialists and geotechnical advisors are given in Site Investigation Steering Group (1993).

Effective communication between all parties involved in the construction process is vital. This includes the site investigation as well, because without accurate and up-to-date information, how can an investigation be designed to answer the questions listed earlier? The *Construction (Design and Management) Regulations* 2007 (known as the CDM Regulations, see Chapter 17) formalize this ethos and require clients and designers to work with all parties from an early stage so that safe methods of work can be planned and adopted. This means that clients and designers must provide the designer and manager of the site investigation with details of the proposed works (e.g., location, depth and size of excavation, support methods). Without these details, it is difficult to plan an investigation on a rational basis.

6.4 STAGES OF SITE INVESTIGATION

A site investigation includes all the activities required to gather the necessary data about the site and should consist of a number of stages, listed below:

1. Desk study
2. Site reconnaissance
3. Ground investigation
4. Reporting

To many nongeotechnical specialists, the ground investigation is perceived to be the "essence" of a site investigation. In fact the ground investigation (which can involve trial pits, borings, in situ testing, and laboratory testing) aims only at determining ground and groundwater conditions at the site. If other stages (especially the desk study) are neglected, inadequate investigations may result.

6.4.1 Desk study and site reconnaissance

The desk study and site reconnaissance (sometimes called a walkover survey) are essential in any investigation—their importance cannot be overestimated. Unfortunately, they are sometimes overlooked or considered irrelevant.

The desk study is a review of all available information relevant to the proposed project including geological and hydrogeological maps, aerial photographs (if available), records of construction on nearby sites or those where similar soil and groundwater conditions were encountered; records of nearby wells, boreholes and springs; and records of mining or previous use of the site. Sources of information for desk studies are given by Dumbleton and West (1976) and in Chapter 3 of Clayton et al. (1995).

The site reconnaissance is usually carried out after the desk study has been largely completed. It comprises a walkover survey of the site to gather further information on the site surface conditions and access for the ground investigation, any exposed geological, groundwater or surface water features, and the nature of the areas surrounding the site.

Clayton et al. (1995) point out that both the desk study and site reconnaissance can provide large amounts of useful information at low cost—they are by far the most cost-effective stages of site investigation. They are essential to allow efficient design of the ground investigation; failure to anticipate any predictable groundwater problems could result in poor or inadequate investigations, leading to potential problems during construction.

The desk study and site reconnaissance can also be used to gather information about the surroundings to the site, perhaps in areas where access for ground investigation cannot be obtained. For groundwater lowering projects, this is particularly useful to help determine whether there will be any adverse side effects of dewatering (such as ground settlement or derogation of water supplies, see Chapter 15) outside the site. Brassington (1986) recommends that a survey of nearby groundwater supplies be carried out to aid the assessment of the risk of derogation to water users.

The problems which may result if the desk study and site reconnaissance are neglected can be illustrated by two case histories.

1. On a project through variable glacial soils in northern England, a new sewer was laid in an open-cut trench to replace an existing sewer, constructed a few decades earlier. The new sewer was generally laid parallel to the old one, apart from one section in which the old sewer took a circuitous "dogleg" route between manholes. The new sewer was to take the obvious straight-line route between the manholes. During construction of this section, severe groundwater problems were encountered, including a flowing artesian aquifer (see Section 3.4) immediately beneath the base of the excavation. Work was further hampered when old abandoned sections of steel sheet-pile trench supports (probably dating from the construction of the old sewer) were encountered. With hindsight, it is fairly obvious that the old sewer was originally intended to take the direct route between the

manholes but groundwater problems forced them to abandon work and reroute the sewer. The new sewer was eventually completed, but a desk study of old construction records may have allowed different methods to be adopted from the start, thus avoiding cost and time delays.

2. A small sewerage project involved a wellpoint system to allow the construction of a manhole. The manhole itself was excavated in sand and gravel and was successfully dewatered. However, significant damage occurred to several neighboring structures as a result of ground settlement. Subsequent investigations showed that the damaged buildings were founded on an extensive deposit of compressible peat. Although ground investigation at the manhole site did not encounter any peat, a desk study review of geological maps would have revealed the presence of peat beneath the surrounding areas. This highlights the risk of damaging settlements and would have allowed appropriate mitigation measures, to reduce the effect of dewatering on nearby buildings (see Section 15.4), to be included in the project.

6.4.2 Ground investigation

There are numerous techniques available to physically investigate a site. It is rarely obvious at the start of an investigation which methods will be suitable to gather the information needed by the project designers. Therefore, it is preferable that all but the smallest ground investigations be carried out in phases. The first phase may consist of an initial pattern of boreholes and testing across the whole site. Preliminary results from the first phase will allow second and subsequent stages to be designed; such stages may include more closely spaced boreholes in areas where ground conditions are unclear, or perhaps specialist testing such as a pumping test.

Ground investigation usually involves the use of some combination of the following methods: boring, drilling, probing, and trial pitting; in situ testing; geophysics; and laboratory testing. These ground investigation methods are briefly outlined below, mainly in relation to British practice as outlined in BS 5930:1999 amended 2010 and BS 1377:1990. Factors particularly relevant to investigations for dewatering projects are highlighted.

In Britain, one of the most common forms of boring is still light cable percussion drilling, colloquially known as "shell and auger" drilling (Figure 6.1a). Soil samples may be recovered from the borehole (for description of soil type), or certain types of in situ tests may be performed within the borehole. A key point to note is that at various stages during drilling, water may be added to the borehole by the driller, or may be removed by the action of the boring tools. This can lead to natural groundwater levels and inflows being masked during boring operations.

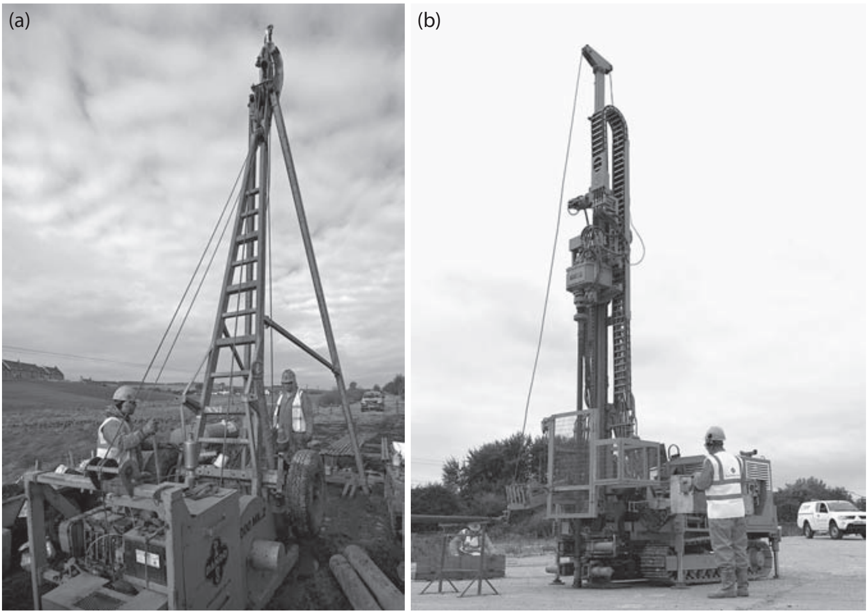


Figure 6.1 Drilling and boring methods used in site investigation. (a) Light cable percussion boring rig. (b) Rotary drilling rig. (Courtesy of ESG Soil Mechanics, Burton on Trent, U.K.)

Rotary drilling (using a water or air-based flush medium with polymer or foam additives) is also widely used (Binns 1998), particularly in relatively intact rock strata, but also in uncemented drift deposits. Rigs are typically mounted on four-wheel-drive tucks or tracked base machines (Figure 6.1a). Core samples can be recovered from the boreholes (for soil and rock description), and certain types of in situ tests should be carried out. Because the borehole is generally kept full of the flush medium, groundwater levels and inflows can be difficult to determine during drilling.

The older technique of wash boring is rarely used in Europe but is still used in countries where labor is cheap. The basic rig is a winch tripod. The associated equipment consists of an outer pipe with a chisel bit at the lower end and a swivel head at the upper end of the wash pipe and incorporating a water pressure hose connection with a weight for driving the casing into the ground. A pump passes water down the wash pipe to slurify the soil at the bottom of the outer casing. The return washings are not regarded as reliable for the identification of soil types—although recordings of wash-water color changes should be noted. This method is suitable for use in sands and silts, but progress in clayey soils is likely to be slow. Groundwater levels and inflows are masked in a similar way to rotary drilling.

Hollow-stem continuous flight augers are suitable for use in cohesive soils but are of limited use in water-bearing granular soils; indeed, in granular soils, this technique is often unworkable. The drilling spoil brought to ground surface gives only an approximate indication of soil types and horizons. Drive-in samplers can be inserted through the hollow stem to obtain strata samples at convenient depth intervals.

As an alternative to boring or drilling, in recent years, probing methods have been developed. A wide range of equipment exists and are used, but the common objective is to determine a profile of penetration resistance with depth. Most methods were developed as a low-cost and rapid alternative to drilling and boring. Two of the most commonly used methods are dynamic probing and static probing by the "cone penetration test" (commonly known as the CPT).

Dynamic probing involves a percussive action to drive the probe into the ground, producing output in the form of blows per unit depth of penetration; for example, see, the work of Card and Roche (1989). Window sampling is a variant on the dynamic probing method that allows soil samples to be obtained via sampling tubes driven into the ground.

Static probing by CPT (also known as "Dutch cone" testing after the country where the method was developed) is more sophisticated than dynamic probing. The cone is pushed continuously into the ground, using reaction from the test truck, producing an output of resistance against depth; see the work of Meigh (1987) and Lunne et al. (1997). Piezocone testing is a variant of the CPT method, in which pore water pressures are measured in addition to resistance parameters; this can allow estimates of permeability to be obtained in low-permeability soils.

Trial pitting is a simple and widely used method for the investigation of shallow strata. A pit is dug, exposing the subsoil for inspection and sampling. Groundwater inflows and seepages can normally be clearly identified. During excavation, it may be possible to form an opinion as to appropriate methods of full-scale excavation, if relevant. Trial pits are normally dug by mechanical excavators (Figure 6.2). Small backhoe loaders can normally excavate to a depth of around 3 m, and larger excavators may be able to work to a maximum depth of around 5 m when working from the ground level. Trial pitting is a potentially hazardous exercise. Trial pits of greater than 1.2 m depth should only be entered if adequately shored and supported. Even pits of less than 1.2 m depth may be unstable. Each pit should be assessed before entry, and if any doubt exists, the pit should not be entered. Soils can be described from the surface, and samples can be taken from the spoil in the excavator bucket. In difficult locations or where ground disturbance must be kept to a minimum, it may be possible to excavate pits by hand excavation. However, this method is very slow, and such excavations must not be taken deeper than 1.2 m without using timbering or some form of proprietary side

Figure 6.2 Trial pitting using a mechanical excavator.

support system. Safety in pits and trenches is discussed by Irvine and Smith (2001).

Further details on all these methods can be found in Clayton et al. (1995). Their relative merits are outlined in Table 6.1. The ground investigation stage also includes the installation and monitoring of groundwater observation wells; this is discussed further in Section 6.6.

Various methods of in situ testing can be carried out as part of boring, drilling, probing, and trial pitting. The most relevant of these to groundwater lowering works are permeability tests; these are described in Section 6.7.

Geophysical methods are sometimes used in investigations for civil engineering works, but these methods are much more widely used in the oil, mineral extraction, and water resource fields. In general, geophysical methods are used to provide information on changes in particular properties of strata beneath a site and can be used to provide information between widely spaced boreholes. The use of boreholes in combination with geophysics is important because the borehole data can be used to "correlate" or "calibrate" the geophysical results for the site in question. Geophysical methods used for civil engineering investigations are described in the work of Clayton et al. (1995), Ch. 4, and McCann et al. (1997). Geophysical methods used in hydrogeological and water resource investigations are described by Barker (1986) and Beesley (1986). On a number of occasions,

Table 6.1 Advantages and disadvantages of methods of boring, drilling, probing, and trial pitting

Method	*Advantages*	*Disadvantages*
Drilling and boring	Suitable for a wide range of soils Allows soil and groundwater samples to be obtained Can allow in situ permeability tests to be carried out Allows observation wells (standpipes and standpipe piezometers) to be installed	Progress can be difficult if cobbles or boulders are present Some methods require specialist equipment
Probing	Provides information on soil profile Piezocone can provide information on soil permeability Can allow simple standpipe observation wells to be installed Some methods allow soil samples to be obtained	Does not provide soil or groundwater samples Some methods require specialist equipment Needs to be used in conjunction with boreholes to enable correlation of soil types Penetration depth is limited in stiff materials and coarse granular soils Does not allow installation of standpipe piezometers
Trial pitting	Suitable for a wide range of soils, including very coarse soils Allows stability and ease of excavation of soils to be directly observed Requires no specialist equipment Allows soil and groundwater samples to be obtained Can allow simple standpipe observation wells to be installed	Depth limited to around 5 m May be difficult to progress below groundwater level in unstable soils Does not allow installation of standpipe piezometers

when geophysics has been used in investigations, the results have been perceived to be disappointing or inconclusive. This is probably a reflection on an inappropriate choice and specification of method, rather than a systematic drawback with the use of geophysics. To get the most out of geophysical surveying, it is essential that engineering geophysicists are involved at an early stage of planning and thereafter; otherwise, the method will not achieve its potential.

Samples of soil or rock obtained from boreholes or trial pits may be tested in the laboratory. The purpose of testing can be as an aid to soil description and classification, or to determine soil properties for engineering design. Properties routinely tested for include strength, compressibility, permeability (see Section 6.7), and chemical characteristics. Samples of groundwater recovered during investigation may also be chemically tested in the laboratory.

6.4.3 Reporting

To be useful to the designers and managers of the project, the site investigation must be reported in an organized, concise, and intelligible manner. Ideally, reporting should be carried out by geotechnical specialists who have been involved with the investigation since its inception. The minimum reporting requirement is for a "factual report" which presents the data gathered during the desk study and site reconnaissance, including the borehole logs, trial pits logs, test results, and groundwater monitoring data from the ground investigation. Such reports do not usually comment directly on the implications of the data gathered.

Eurocode 7 (BS EN 1997-1:2004) sets out the requirements for a Ground Investigation Report (GIR) which is analogous to a factual report, perhaps with a slightly wider scope to include some basic interpretation of data. Eurocode 7 specifies that the GIR should include:

- A presentation of the available geotechnical information including desk study information
- A factual account of all field and laboratory investigations
- A geotechnical evaluation of the information, stating the assumptions made in the interpretation of the test results
- A statement of methods adopted (citing the relevant standards)
- All relevant information on how a direct assessment of how any derived values were determined, including any correlations used; and
- Any known limitations in the results

In many cases, particularly on large or complex projects, an "interpretative report" is produced in addition to the factual report. Again, written by geotechnical specialists, this should review the ground and groundwater conditions at a site. It should include discussion of the effect of the anticipated conditions on the proposed design and construction methods. At the time the interpretative report is produced, the project design may not be finalized, but the report should discuss the geotechnical aspects of the full range of design options current at that stage. If particular potential problems in design and construction are highlighted, one of the report's conclusions may be to recommend further or specialist investigations.

Eurocode 7 (BS EN 1997-1:2004) sets out the requirements for a Geotechnical Design Report (GDR) which is a form of interpretative report. Eurocode 7 specifies that the GDR should include:

- A description of the site and surroundings
- A description of the ground conditions
- A description of the proposed construction
- Design values of soil and rock properties, including justification

- Statement of the codes and standards applied
- Statements of the suitability of the site with respect to the proposed construction and the level of acceptable risks
- Geotechnical design calculations and drawings
- Recommendations for foundations and ground treatments; and
- Recommendations for supervision and monitoring during construction

6.5 DETERMINATION OF GROUND PROFILE

Any investigation will need to identify the nature, depth, extent, and orientation of the strata beneath the site. Collectively, these parameters describe the "ground profile," normally based on the results of trial pits, boreholes, geophysics, and probing. Determination of the ground profile is an essential part of developing the groundwater conceptual model (see Section 7.4) needed to allow dewatering design.

It is essential that boreholes penetrate to adequate depth. The presence of confined aquifers or of localized zones of high permeability beneath excavations is a significant risk for many excavations. Boreholes in the area of the excavation should penetrate to a depth of 1.5–2 times the depth of the excavation. There have been several cases in which excavations failed due to base heave (see Section 4.6) when investigation boreholes were not taken to adequate depths but were terminated a few meters below the proposed formation level. The failure was caused by confined aquifers below the formation, undetected during investigation. Such problems are frustrating because if the boreholes had been deeper and had detected the aquifer, groundwater control (using relief wells) would have been simple and cost-effective. As it was, major cost and time delays resulted. The only case when boreholes shallower than the recommendation can be tolerated is if the desk study clearly indicates that impermeable soils are present to considerable depth below formation level.

In addition to soil descriptions, groundwater level information and permeability test results should help identify which strata are water-bearing (and may act as aquifers) and which strata are of low-permeability (and may act as aquitards and aquicludes). Compressibility test results may help identify any strata that may give rise to significant groundwater lowering–induced settlements.

6.6 DETERMINATION OF GROUNDWATER CONDITIONS

In many investigations, the observations of groundwater conditions are totally inadequate, providing little concrete information on groundwater levels. Accurate knowledge of the likely range of groundwater levels and pore water pressures in the various strata is essential for the design of dewatering systems. This section will describe the types of groundwater observations that may be taken during investigation and will discuss their various merits and limitations.

6.6.1 Water level observations in trial pits and borings

The easiest and most common form of groundwater level observations are those taken in trial pits and borings. Unfortunately, there are two important limitations to the accuracy of readings obtained in this way:

1. The natural groundwater inflows and levels may be masked or hidden by the excavation or boring method, particularly if water is added or removed from the pit or borehole, or if drilling casing seals off inflows of water.
2. For a pit or borehole to show a representative groundwater level, sufficient water must flow into the pit or borehole to fill it up to the natural groundwater level. In soils of moderate or low-permeability, it can take a long time for the water level in the pit or borehole to come into equilibrium with the natural groundwater level. Most observations do not allow sufficient time for equilibrium and, therefore, may report unrepresentative water levels.

Trial pits offer a simple way to observe shallow groundwater conditions. The size of the pit allows direct visual observation of inflows and seepages (it may be possible to categorize these as "slow," "medium," or "fast" seepages on a subjective basis). The location of the seepages in relation to the soil fabric and layering can provide information on the relative permeability of various strata—this can be a useful way of identifying perched water tables. The disadvantage of trial pits is that, because of their relatively large volume, a significant volume of water has to enter the pit to fill the pit up to the natural groundwater level. This process of equalization may take several days in soils of moderate permeability. The pit could be left open and monitored daily but, for safety reasons, trial pits are rarely left open for long periods of time. In general, groundwater levels in trial pits may not be representative of natural groundwater levels.

In British site investigation practice, the most common form of groundwater observations are those recorded by the drilling foreman during light cable percussion drilling. Each time groundwater is encountered, these records should comprise:

1. The depth at which water is first encountered (known as a "water strike").
2. A description of the speed of inflow (e.g., slow, medium, or fast). Boring is normally suspended after a water strike and groundwater

levels observed to record the increase (if any) in groundwater levels. Ideally, monitoring should continue until the water level in the borehole stabilizes, but oftentimes, the increase in water level is recorded for a fixed period (often 20 min) only, before boring recommences.

3. The depth at which the groundwater inflow is sealed off by the temporary drilling casings.

Additionally, the drilling foreman should record whether water was added to the borehole during drilling and should record the water level in the borehole at the start and end of the drilling shift (together with the corresponding depth of borehole and casing at that time). All groundwater details are recorded on the driller's daily record sheet and should appear on the final borehole log.

When reviewing water levels recorded during boring, the following points must be noted:

1. In all but the most permeable soils, observing the water level increase for a short period after a water strike may not allow sufficient time for the natural groundwater level to be apparent. In many cases, the water level recorded after a water strike will be lower than the actual groundwater level in that stratum.
2. A significant increase in borehole water level after a water strike may indicate the presence of a confined aquifer, particularly if the water strike occurs just after the borehole has passed from a low-permeability stratum into a high-permeability one. However, smaller increases in water levels may be observed, even in unconfined aquifers. This occurs when the driller has drilled a short distance below the water table before noticing the inflow. The drilling action will have removed some water from the borehole and, when drilling stops, the water level will rise up to the natural level.
3. Because of the speed of boring, when drilling through soils of very low to moderate permeability (such as clays, silts, and silty sands) a water strike may not be noticed at all by the driller. The spoil from the borehole will be damp or moist, but there will not be time for free water to enter the borehole. There have been cases in which groundwater inflow was not recorded in investigation borings through strata of silty sand, yet excavation works encountered groundwater and needed significant dewatering. This error could have been avoided by installing and monitoring observation wells, which allow time for equalization of water levels.
4. The boring process will inevitably alter the water level in the borehole so it is not representative of that in the surrounding soil. The action of the drilling tool will remove water, and the driller may be deliberately

adding water as part of the drilling process. The water level in the borehole at the end of the shift is likely to be very unreliable, being highly influenced by the recent drilling activities. If drilling work is done on a day shift basis, the water level at the start of shift (next morning) may be more reliable as the borehole water level will have begun to equalize overnight. Even so, the start of shift level may still be unrepresentative, especially in low-permeability soils, where longer periods may be necessary for full equalization.

Observations during rotary drilling are generally unsatisfactory, because during drilling, the borehole is either kept topped up with water (if water-based flush fluids are used) or water is continually blown out of the borehole (if air-based flush fluids are used). It can be difficult to detect discrete groundwater inflows during drilling. As with boring methods, the water levels at the start of a shift tend to be more reliable than those observed at the end of the shift.

The principal drawback which affects all groundwater observations during trial pitting or boring is that, at best, they only give "spot" readings of conditions at the time of the investigation. As was discussed in Chapter 3, in general, groundwater levels are not constant but will vary with the seasons, with long terms trends (such as drought) or in response to external influences. Observations in trial pits and boreholes cannot give information on potential variations and (depending on the time of year when they were drilled) may not indicate the highest groundwater level that can occur at a site. This type of information can only be obtained by the installation and monitoring of observation wells of some sort.

Table 6.2 summarizes the advantages and disadvantages of methods of determining groundwater levels.

6.6.2 Observation wells

An observation well is an instrument installed in the ground at a specific location to allow measurement of the groundwater level or pore water pressure. When determining groundwater levels, observation wells have a number of advantages over observations during boring:

1. Because they are long-term installations, appropriately designed observation wells can allow observation of equilibrium groundwater levels, even in very low-permeability soils.
2. Observation wells can be installed as piezometers to record water levels or pore water pressures in a specific stratum.
3. Observation wells can be monitored for long periods of time, to observe the variations in groundwater level at a site.

Table 6.2 Advantages and disadvantages of methods of determining groundwater levels

Method	*Advantages*	*Disadvantages*
Observations in trial pits	Commonly carried out Allows seepage into excavation to be observed directly May allow perched water tables to be identified	Limited to shallow depth Standing water levels may be unrepresentative unless pit is left open long enough for equalization to occur
Observations during boring	Commonly carried out Normally sufficient to identify water inflows from major water-bearing strata	True groundwater seepages and levels may be masked by drilling action and by the addition and removal of water Seepages in soils of low to moderate permeability may not be identified Insufficient time is normally allowed for water levels to equalize to natural levels
Observations during rotary drilling	Commonly carried out	Groundwater seepages and levels tend to be masked by the presence of flush medium
Observation wells—standpipes	Cheap and simple to install in boreholes Useful in simple unconfined aquifers Allows monitoring of groundwater levels in the long-term after boring is completed	Not appropriate for sites with confined aquifers, multiple aquifers, or perched water tables May need to be purged or developed before use May need to be protected from vandalism and damage
Observation wells—standpipe piezometers	Relatively straightforward to install in boreholes Allows groundwater levels in specific strata to be observed Can be used on sites with complex groundwater conditions Allows monitoring of groundwater levels in the long-term after boring is completed	Closer supervision of installation is needed than with standpipes May need to be purged or developed before use May need to be protected from vandalism and damage
Specialist instruments—pneumatic and electronic piezometers	Can allow accurate pore water pressure measurements in low- and very low-permeability soils Can be read remotely Allows monitoring of groundwater levels in the long-term after boring is completed	Installation and calibration in boreholes can be complex; close supervision is advisable Readings may not be straightforward to interpret May need to be protected from vandalism and damage

The two most commonly used devices for monitoring groundwater levels in permeable soils are standpipes and standpipe piezometers.

The simplest form of observation well is the standpipe (see Figure 6.3). This consists of a small-diameter pipe, of which the bottom section (usually at least 1 m in length) is perforated or slotted, with the base plugged. The pipe is installed in the center of a borehole and sand or gravel placed around the pipe and, if necessary, tamped into place. Backfilling should cease at a depth of about 0.5 m below ground level and the remainder of the hole sealed using puddled clay or bentonite/cement and capped off with concrete to prevent surface or rainwater from entering the borehole. It is advantageous to haunch the concrete to help shed the surface water. Unless a special protective cover is required, the pipe should project about 0.5 m above ground level and be provided with a suitable cap or threaded plug. In urban areas, it is essential that the cover or capping arrangement is secure enough to resist vandalism. Some designs of covers (known as "stop-cock" covers) can be installed flush with the ground surface. These covers are sometimes preferable in vandal-prone areas because they are unobtrusive and may not attract the attention of vandals.

Plastic tubing such as PVC or high-density polyethylene (HDPE) is an ideal material for standpipe tubing. Typically supplied in 3-, 5-, or 6-m-long pieces with threaded connections, it can readily be sawn to the desired length if necessary and joined using PVC couplings and solvent cement. The perforated lengths of pipe are usually supplied in 1.5 m lengths and are predrilled or preslotted. If necessary, the plain pipe can be slotted on-site using a hacksaw but it should be noted that the total area of perforations should be at least twice the cross-sectional area of the standpipe. The water level in a completed standpipe can be measured using a dipmeter (see Section 16.3). The preferred internal diameter for standpipe tubing is approximately 50 mm; this enables water samples to be taken and allows a small airline to be used to flush out the standpipe if it becomes blocked.

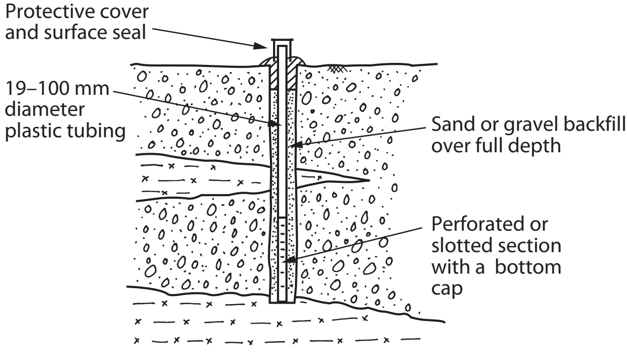


Figure 6.3 Typical standpipe installation.

Smaller diameter tubing is sometimes used, but the minimum acceptable internal diameter is usually 19 mm, because this is the smallest size down which many commercial dipmeters can pass.

A standpipe is simple and cheap to install, but it is only a basic instrument. The standpipe will respond to pore water pressures in water-bearing strata along its entire depth. This is acceptable if the standpipe is used in a simple unstratified unconfined aquifer, in which the total head is constant with depth. However, if a standpipe is installed in a layered aquifer system, water can enter from more than one water-bearing layer. If the groundwater levels are different in each layer (e.g., a main water table and a perched water table) the standpipe will show a "hybrid" water level between the two true water levels.

Standpipes are not suited to use in layered or complex groundwater regimes. In such cases, it is necessary to use a "piezometer" where the instrument is sealed into the ground so that it responds to groundwater levels and pore water pressures over a limited and defined depth only. The most common type of piezometer is the standpipe piezometer.

Figure 6.4 shows typical construction details for standpipe piezometers. The aim is to produce a "response zone" of sand or fine gravel at the level of the stratum in which the groundwater level is to be observed. Rigid PVC

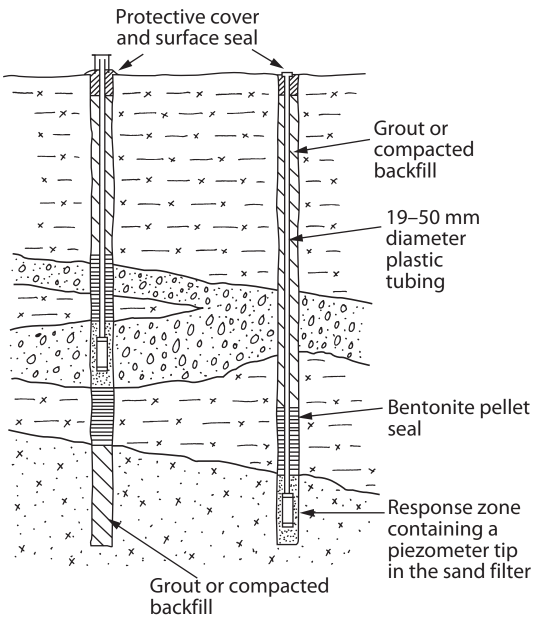


Figure 6.4 Typical standpipe piezometer installations. Two piezometers are shown, each with its response zone and piezometer tip in a different water-bearing stratum.

or HDPE tubing is installed in the borehole in a similar way to a standpipe, with a porous or perforated "piezometer tip" located in the center of the response zone. Grout seals above and below the response zone ensure that water can only reach the tip from the desired stratum. As with a standpipe, water level readings could be taken with a dipmeter.

Installation of standpipe piezometers is more complex than for standpipes and should be carried out with care. It is essential that the seals are effective, otherwise water may leak into the response zone from the strata above or below. If clay backfill is used, it must be adequately compacted (e.g., with the drill rods or shell) to reduce the risk of later settlement. Where grout is used to backfill parts of the boreholes, it should be cement–bentonite grout of the appropriate consistency. A layer of bentonite pellets should be placed between the grout seal and the sand filter in the response zone, to avoid the sand becoming contaminated with grout (if pellets are not available, bentonite balls will have to be made up by hand). Once the lower bentonite seal is in place and has had time to swell, it is good practice to flush out the dirty water in the borehole and replace it with clean water before installing the sand filter.

A "generic" specification of the grading of sand filters is not possible. However, for guidance, a filter consisting of a clean, well-graded sand and gravel, with only a small proportion of fine to medium sand, is suitable for soils with some clay or silt content. For a fine sand soil, the filter should consist of coarse sand or coarse sand and gravel with not more than a small percentage of medium sand. Local material may have to be used, but it is essential that the filter material is free from clay and silt. Bentonite pellet and grout seals are installed above the sand filter. The tubing should be capped off at ground level with a secure cover or headworks.

The "piezometer tip" typically consists of a porous plastic element or a porous ceramic element (sometimes known as a "casagrande element"); tips are generally 150–600 mm in length. It is good practice to soak the filter sand and ceramic element (if used) in water before installation—this helps avoid any air from being trapped in the system and speeds up the process of equilibration between the piezometer and the natural groundwater level.

It is possible to install two or more piezometer tips (each in its own response zone and separated by grout seals) in one borehole. If this is being contemplated, it is essential that it is carried out by experienced personnel and is carefully supervised. This is awkward work in all but the largest boreholes and there is always the risk that the installation of the second tip and seals will affect the piezometer already installed. If possible, single piezometer installation in each borehole should be used, purely because the water level readings will be easier to interpret, with no worry of water leaking between response zones.

In many investigations for groundwater lowering projects, piezometers will be installed in relatively permeable soils, in which the water level inside the piezometer will respond rapidly to changes in the pore water pressure in the soil. However, piezometers in soils of moderate to very low-permeability may respond slowly to changes in pore water pressure. This is because a finite volume of water must flow into or out of the piezometer to register the change in pressure. This leads to a "time lag" between changes in pore water pressure in the soil and the registering of that change in the piezometer. The time lag is greater in soils of lower permeability and is greater for piezometers in which larger volume flows are needed to register pressure changes.

In a standpipe piezometer, the prime factor controlling the equilibration rate is the internal diameter of the tubing; the smaller the diameter, the shorter the time lag and the quicker the piezometer will respond to pressure changes. In soils of low to moderate permeability, it is normal to specify the internal diameter of the tubing to as small as possible (19 mm is the lower practicable limit to allow monitoring by dipmeter). However, in permeable soils such as sand and gravels, the equilibration rate will tend to be rapid, and 50 mm diameter tubing could be used, allowing greater flexibility for sampling or flushing out of the piezometer.

A defining feature of observation wells is that (provided they are adequately protected from damage or vandalism) they can be used to observe groundwater levels long after the main ground investigation is complete. This can allow natural changes in groundwater levels to be determined. However, this requires the instruments to be monitored for extended periods, and the practicalities of this are sometimes overlooked. If readings are to be taken manually, this will have to be included in the site investigation plan and associated costings. In remote or inaccessible sites, it may be appropriate to use datalogging systems (see Section 16.6) to record groundwater levels and to reduce the cost associated with regular visits by personnel.

6.6.3 Other methods for determination of groundwater levels and pore water pressures

There may be occasions, especially in soils or rocks of low or very low-permeability, when, because of the finite volume of water which must flow to register a change in the water level, the time lag is so great that the equilibration rate of a standpipe piezometer is too slow to give useful readings. In such cases, it may be appropriate to use specialist "rapid response" piezometers. Such instruments are characterized by the very small volume of water which must flow into or out of the sensor to record a change in pressure; they have been used successfully to observe pore water pressure changes in silts, clays, and laminated soils.

Before the 1990s, the most common form of rapid response piezometers were hydraulic and pneumatic piezometers, in which the movement of small volumes of fluid (water or air, respectively) in closed systems inside the instrument is used to balance external water pressure. Measurement of the internal pressure in the system linked to the instrument allowed the external groundwater pressure to be determined.

In recent years, the vibrating wire pressure transducer (often known as a vibrating wire piezometer or VWP) has become the most widely used instrument type in rapid response piezometers. These instruments contain a metal diaphragm in hydraulic connection with the groundwater. Inside the instrument, a taut wire is stretched between the diaphragm and a stable datum. When the instrument is read, it is "plucked" by passing a controlled frequency electrical pulse along it. The taut wire resonates at a frequency related to its tension, which can be related to the deflection of the diaphragm and hence the water pressure on the diaphragm. The VWP instruments are linked to the surface by a small diameter cable, which allows the instruments to be linked to dataloggers (see Section 16.6) or to specialist portable readout units. Such specialist instruments must be specified, installed, and calibrated with care; instrument manufacturers can often provide useful advice.

Traditionally, rapid response instruments were installed at a specified level in a discrete filter sand response zone in a similar way to a standpipe piezometer, but instead of an open pipe, these instruments are connected to ground level by a cable (Figure 6.5a). This approach is effective but time-consuming. An alternative approach, known as the "fully grouted method" has been developed wherein the VWP instruments are installed in the borehole surrounded by cement–bentonite grout, with no sand response zone (McKenna 1995; Contreras et al. 2007).

The fully grouted method of installation of VWP piezometers takes advantage of the fact that these instruments require only a very small volume of equalization (10^{-6} to 10^{-7} m^3) to respond to water pressure changes. This means that an appropriate cement–bentonite grout is able to transmit this small volume over the short distance which separates the VWP instrument from the ground around the borehole. Studies have shown that provided the grout mix is no more than two orders of magnitude more permeable than the surrounding ground, inaccuracies in the measured pressure caused by the presence of the grout will not be significant.

Installation involves lowering the VWP instrument into the borehole, often attached to a sacrificial grout pipe. The instrument is set at a specified level in the borehole (Figure 6.5b). This must be done accurately so that the pressures measured by the instrument can be related to groundwater heads. The borehole is then backfilled with a carefully controlled cement–bentonite grout mix. Care should be exercised to ensure that air bubbles are not trapped in the grout around the VWP sensor—such bubbles would affect the pressure response of the instrument. To avoid air from being trapped against the VWP diaphragm, the instruments are sometimes

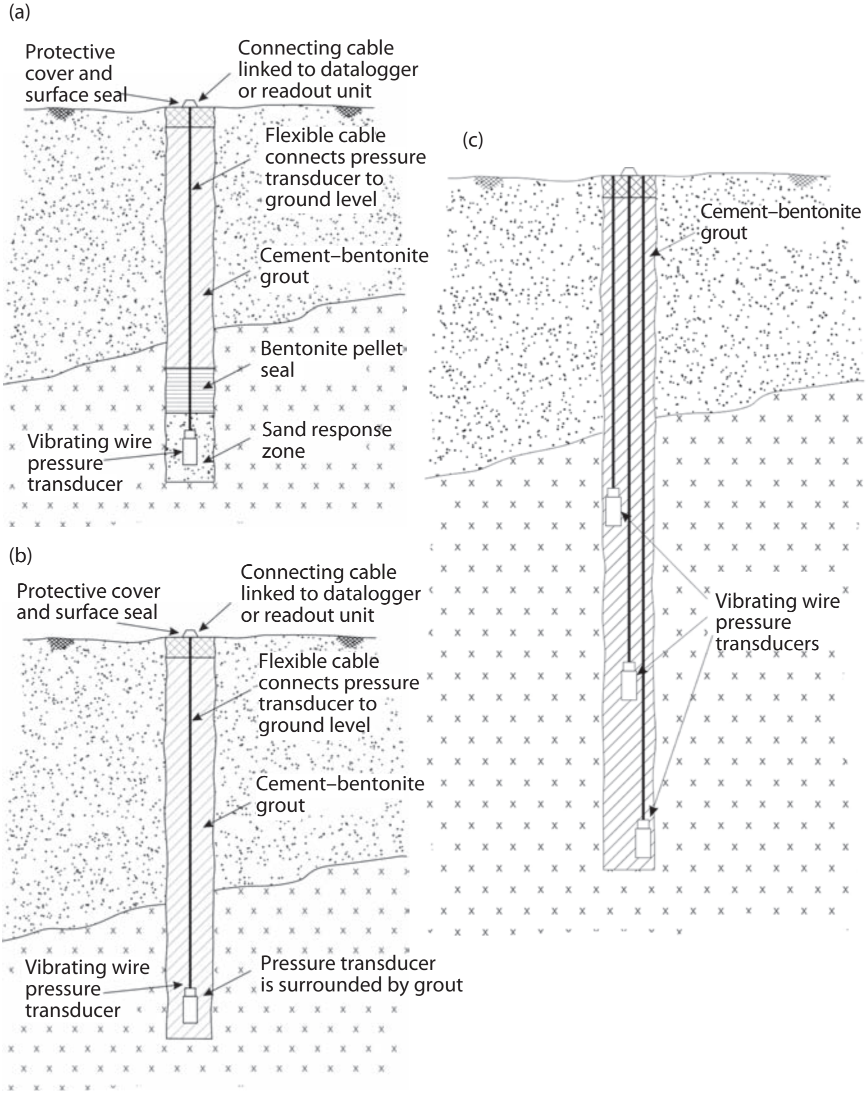


Figure 6.5 Installation of VWPs. (a) Installation in sand response zone. (b) Installation in fully grouted borehole. (c) Installation of multiple piezometers in fully grouted borehole.

installed with the diaphragm facing vertically upward or with the space in front of the diaphragm prefilled with a flexible void filler.

The fully grouted method also has the advantage that it can be used for the installation of multiple or “nested” piezometers within a single borehole (Figure 6.5c). This can be useful when there is a requirement to measure groundwater heads at different depths, such as when vertical groundwater flow is of concern.

At ground level, the cables can be terminated (to be read later using a portable readout unit) or can be connected to a datalogger system (see Section 16.6) to allow regular readings at preprogrammed intervals. The datalogger unit can be located some distance from the piezometer itself, being connected by cable. This allows these instruments to be used where there is no permanent surface access for reading; with suitable grout seals, they have been used in boreholes beneath rivers and lagoons, with the datalogger located on the shore.

These instruments do not record the total head but the pore water pressure (above the level of the instrument). To interpret readings from the instruments correctly, the precise level at which the instrument was placed must be recorded during installation. The groundwater total head is the sum of the elevation of the instrument and the pressure head in the instrument (Figure 3.4). Determination of total head or piezometric level from pore water pressure readings is described in more detail in Section 3.3.

6.6.4 Groundwater sampling and testing

Knowledge of groundwater chemistry may be required for a variety of reasons (see Section 3.9). In particular, groundwater chemistry can influence how discharge water can be disposed of, or how it may potentially affect dewatering operations. Groundwater samples should be obtained and tested as part of the site investigation.

Where groundwater is encountered during drilling and boring, obtaining a sample is relatively straightforward. The water sample should be taken using a clean sampling bailer as soon as possible after the seepage has begun. If water has been added during boring, this should be bailed out before taking the sample.

Groundwater samples can also be taken from observation wells by use of a sampling pump. The water standing in the observation well has been exposed to the atmosphere and is unlikely to represent the true aquifer water chemistry. Therefore, it is vital to fully "purge" the well before taking a sample. Purging involves pumping the observation well at a fairly steady rate until at least three "well volumes" of water have been removed (a well volume is the volume of water originally contained inside the well liner). Specialist sampling pumps should be used in preference to airlifting because the latter method may aerate the sample, increasing the risk of oxidation of trace metals and other substances. Obtaining a water sample during a pumping test is described in Appendix 3.

In general, water samples should be of at least 1 L volume. The bottles used for sampling should be clean with a good seal and should be filled to the brim with water to avoid air bubbles in the sample. The sample bottle should be washed out three times (with the water being sampled) before taking the final sample.

Samples may degrade after sampling and should be tested as soon as possible after they are taken. Ideally, they should be refrigerated in the meantime. However, even then, the sample may degrade in the bottle (for example, by trace metals oxidizing and precipitating out of solution). Specialists may be able to advise on the addition of suitable preservatives to prevent this from occurring. The choice of sample bottle (glass or plastic) should also be discussed with the laboratory because some test results can be influenced by the material of the sample bottle. Above all, it is vital to use an accredited, experienced laboratory for the chemical testing of water samples.

6.7 DETERMINATION OF PERMEABILITY

Permeability (also known as hydraulic conductivity) is an essential parameter to be determined for the design of groundwater lowering systems. There is a wide range of methods for determining permeability, some of which have been in use since the nineteenth century. The available methods can be classified into four main types:

1. Laboratory testing (including particle size analysis and permeameter testing)
2. Small-scale in situ tests (including borehole, piezometer, and specialist tests)
3. Large-scale in situ tests (including pumping tests and groundwater control trials)
4. Other methods (including geophysics, visual assessment, and inverse numerical modeling)

Although many test methods are available, obtaining realistic values of permeability is far from straightforward. The key problem is that, as was described in Section 3.3, soils and rocks are not homogenous isotropic masses. Permeability is likely to vary from place to place and to be different for different directions of measurement. As was stated by Preene et al. (2000) “Even if it could be obtained, there is no single value of permeability in the ground waiting to be measured.”

If it is accepted that it can be difficult to determine meaningful values of permeability, that should still not deter those involved in site investigation and dewatering design from putting their best efforts into obtaining the most useful values practicable. The following sections outline the characteristics and limitations of the most commonly used methods of determining permeability. The final section in this chapter discusses the relative reliability of the various techniques. Selection of permeability for the design of groundwater lowering systems is dealt with in Chapter 7.

6.7.1 Visual assessment

Every site investigation carried out to modern standards will contain, as part of the borehole logs or trial pit logs, detailed descriptions of the soils or rocks encountered. The description is normally carried out in accordance with closely defined methodologies set out in geotechnical standards. In the United Kingdom, the protocols for description of soils and rocks are described in BS EN 14688-1:2002 and BS EN 14689-1:2003, respectively. Similar protocols exist in the geotechnical standards applicable in other countries.

The purpose of requiring soils and rocks to be described to consistent standards is that it allows an experienced engineer or geologist, reading a borehole log, to be able to glean information on the physical nature of the soil or rock encountered in the borehole. Therefore, it should be recognized that the description of soil or rock contains a lot of information from which it may be possible to infer approximate values for some physical properties of the material, including permeability.

Visual assessment of the permeability of a soil sample is the process of assessing the soil type or grading and, based on experience or published values (such as Table 3.1), estimating a very approximate range of permeability. This method is essential to allow the corroboration of permeability test results. On *every* project, the soil descriptions from borehole or trial pit logs should be reviewed to give a crude permeability range, against which later test results can be judged. Information gathered by the desk study, such as experience from nearby projects, can be useful in this regard.

For example, a soil described as a medium sand might typically be expected to have a permeability of the order of 10^{-4} m/s; certainly, such a soil would be unlikely to have a permeability greater than 10^{-3} m/s or less than 10^{-5} m/s. If permeability tests give results of 10^{-8} m/s, there is clearly some discrepancy. Either the soil description is misleading or the test results are in error or unrepresentative. If this is recognized while testing is still going on, there may be a chance to modify test types or procedures to get better results. Discrepancies of this magnitude are not rare, and visual assessment of permeability can often be a more useful guide to permeability than test results, especially if the latter are limited in scope and questionable in quality.

6.7.2 Inverse numerical modeling

This approach can only be used if extensive groundwater monitoring data (e.g., data from a number of observation wells over a long period) are available. It also requires a thorough understanding of the geological structure and extent of the aquifers in the area. The method involves setting up a numerical model (see Section 7.10) of the aquifer system in the vicinity of the site and running the model with a variety of permeability and other parameters, until an acceptable match is obtained between the model output and the groundwater monitoring data.

This approach is not straightforward and is carried out only rarely. Because a number of parameters, in addition to permeability, will be varied during the modeling, "nonunique" solution may result. In other words, a number of permeability values may give an acceptable fit with the data, depending on what other parameter values are used.

6.7.3 Geophysics

Geophysical methods have traditionally been used extensively in hydrogeological studies for the development of groundwater resources but are used only rarely in investigations for construction projects. The methods available can be divided into two main types:

1. Surface geophysical methods
2. Downhole (or borehole) geophysical methods

These methods do not generally give *direct* estimates of permeability values but can allow indirect estimation of permeability, identification of zones of relatively high or low-permeability, or correlation of permeability values between different parts of a site (MacDonald et al. 1999).

Surface geophysical methods include seismic refraction methods, gravity surveying, electromagnetic surveying, and the most widely used method, resistivity soundings (Barker 1986). These methods measure the variation in specific physical properties of the subsurface environment and apply theoretical and empirical correlations to infer the structure of the ground and groundwater regime. Typical applications include mapping the extent of gravel deposits within extensive clay strata, or locating buried channel features within drift deposits overlying bedrock.

Downhole geophysical methods involve surveying previously constructed boreholes. This is normally achieved by lowering various sensing devices (known as "sondes") into the borehole. Some types of sonde investigate the properties of the aquifer material (this is known as formation logging), whereas others measure the properties of the water in the borehole (fluid logging). The various methods available are described in Beesley (1986) and BS 7022:1988. These methods are generally applicable in boreholes drilled into rock, in which more permeable fissured or fractured zones can sometimes be identified.

Geophysical methods are specialized techniques. Their application and interpretation requires care. In all but the simplest of cases, specialist advice should be obtained from experienced geophysicists.

6.7.4 Laboratory testing: Particle-size analysis

There are a number of empirical methods available to allow the permeability of granular soils to be estimated from analyses of particle size distributions (PSDs) of samples. An American waterworks and sanitary engineer from New England, Allen Hazen (1892, 1900) was the first to propose an empirical correlation for the permeability of a sand from its PSD curve. Probably due to the great simplicity of his "rule," Hazen's rule is still widely used by many of today's geotechnical practitioners; often without due regard to the limitations that Hazen himself stated. His objective was to determine guidelines for suitable sand gradings for water supply filtration. He determined that the D_{10} particle size (called the "effective grain size") and D_{60}/D_{10} (the "uniformity coefficient") were both important factors.

Hazen included allowances for variations in the temperature of the water. However, the temperature of groundwater in the United Kingdom varies little between about 5°C and 15°C; so Hazen's rule used to estimate permeability k may be stated as

$$k = C(D_{10})^2 \tag{6.1}$$

where C is a calibration factor and D_{10} is the 10% particle size taken from the particle size distribution curves (Figure 6.6).

Hazen stated in his work that his rule was applicable over the range of D_{10} particle size 0.1–3.0 mm and for soils having a uniformity coefficient of less than 5. He also stated that (when k is in meters per second and D_{10}

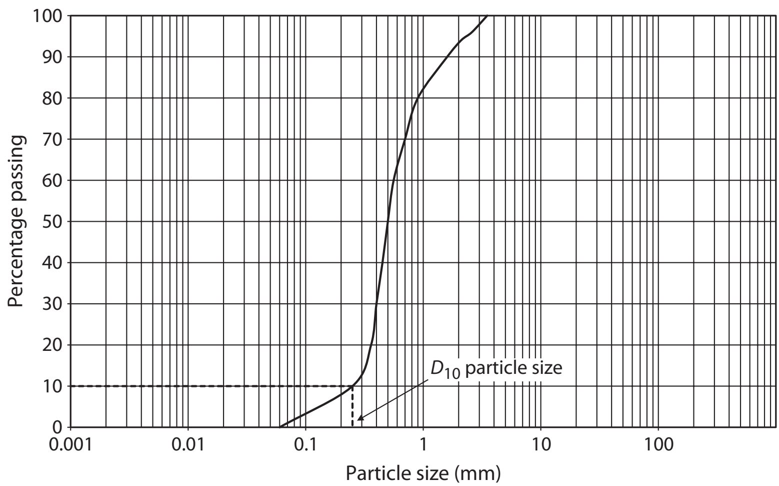


Figure 6.6 Application of Hazen's rule.

is in millimeters) his calibration factor C could vary between about 0.007 and 0.014. In practice, presumably for reasons of simplicity, C is normally taken to be 0.01. It cannot be stressed too strongly that, even within its range of applications, Hazen's rule gives *approximate* permeability estimates only.

Since Hazen, many others—particularly Slichter, Terzaghi, Kozeny, and Rose (all reported in Loudon 1952) and Masch and Denny (reported in Trenter 1999)—have developed expressions for estimating permeability values from grain size distributions of sands. Unlike Hazen, who did not seek to address in situ soils, some have taken account of porosity, angularity of the grains, and specific surface of the grains. None claim to be relevant to soils other than "a wide range of sands."

Loudon (1952) thoroughly reviewed various published formulae and supplemented his review with his own laboratory investigations. He concluded that the error prediction using Hazen's rule could be of the order of ±200% but that Kozeny's formula—which is similar to that of Terzaghi, although more complicated—was to be preferred over various other formulae. Loudon stated that an accuracy of about ±20% can be expected from Kozeny's formula.

He also proposed that his own formula, based on Kozeny, should be used for reasons of simplicity.

$$\log_{10}(kS^2) = a + bn \tag{6.2}$$

where k is the permeability expressed in centimeters per second, n is porosity of the granular soil (a dimensionless ratio, expressed as a fraction not as a percentage), S is specific surface of grains (surface area per unit volume of grains) expressed in square centimeters/cubic centimeters, and a and b are calibration factors with values of 1.365 and 5.15, respectively.

Although the porosity of a sample can be determined in the laboratory, it is virtually impossible to determine the porosity of a sample in situ. This is a limitation on the usefulness of the method from Loudon and other similar works and an explanation for the somewhat erratic results that they sometimes give. They take little or no account of the density and heterogeneity of soils. Standard penetration test N values do give an indication of relative density of granular soils and so may afford some *tentative* indication of porosity. Refer to Appendix 1 for further information concerning Loudon's method.

In the 1950s, in America, the late Professor Byron Prugh researched and developed an empirical method for estimating permeability based on the use of particle size data together with in situ density field measurements. He checked his predictions against field measurements of permeability. Prugh's approach represents a return to the pragmatic coordination of academic and field observations.

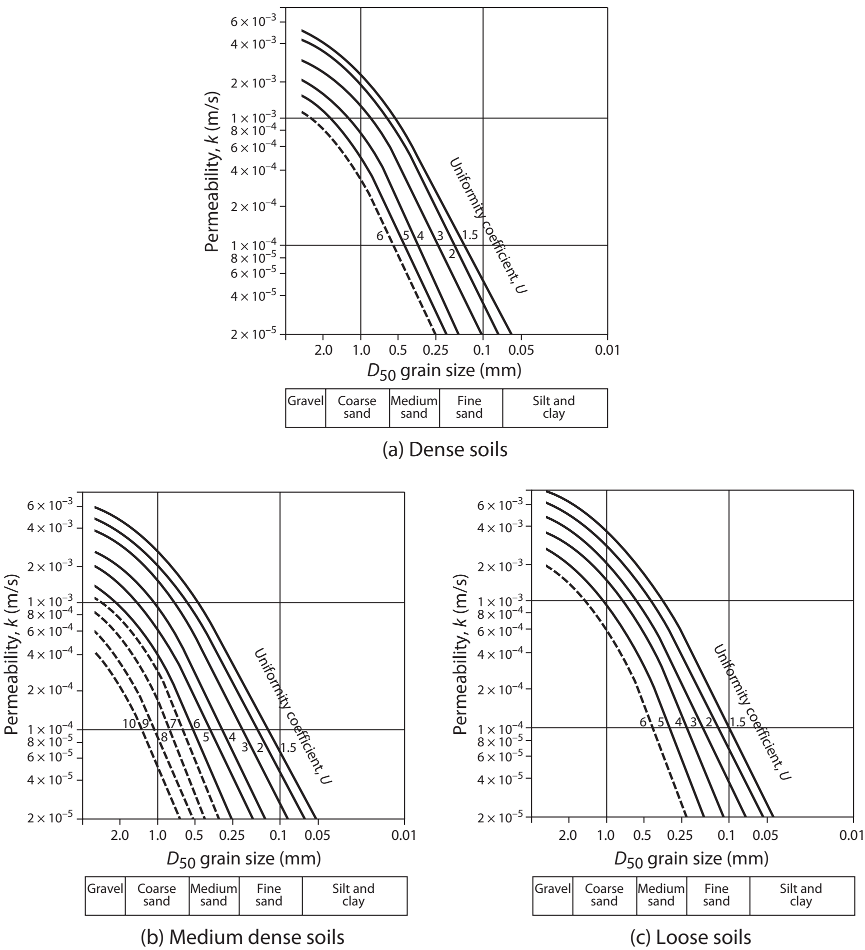


Figure 6.7 Prugh method of estimating permeability of soils. (From Preene, M., Roberts, T.O.L., Powrie, W., and Dyer, M.R., Groundwater control—Design and practice, *CIRIA Report C515*. Construction Industry Research and Information Association, London, 2000. Reproduced by kind permission of CIRIA: www.ciria.org.)

Prugh plotted (Figure 6.7) curves for various uniformity coefficients (D_{60}/D_{10}). The D_{50} grain sizes are plotted on the horizontal axis to a log scale. Permeability is plotted on the vertical axis, also to a log scale. Three separate sets of uniformity coefficient curves were compiled for

1. Dense soils
2. Medium dense soils
3. Loose soils

To use Prugh's curves, first determine whether the soil sample is dense, medium dense, or loose (based on standard penetration test *N* values from borehole logs); then project upward from the D_{50} grain size of the sample onto the appropriate uniformity coefficient curve; from the uniformity coefficient curve, project horizontally to read off the permeability value.

Prugh's data indicate that as the uniformity coefficient increases (i.e., the sample becomes less and less a single-size material), the permeability decreases noticeably. The significance of the Prugh curves, apart from their usefulness, is that of greatly helping the understanding of the interrelationship of various factors (other than D_{10} used in Hazen's rule) affecting soil permeability. His work has been published by Powers et al. (2007) and in CIRIA Reports (Preene et al. 2000). Like others, Prugh did not claim his method to be relevant to soils other than "a wide range of sands."

Irrespective of which method is used to estimate permeability, these approaches all use data from samples recovered from boreholes, rather than tested in situ. This can lead to inaccuracies in permeability assessment, including

1. Any soil structure or fabric present in the in situ soil will be destroyed during sampling and test specimen preparation. Permeability estimates based on the PSD curve of the resulting homogenized sample are likely to be unrepresentative of the in situ permeability. If a clean sand deposit contains laminations of silt or clay, these will become mixed into the mass of the sample during preparation and the PSD curve will indicate a clayey or silty sand. Underestimates of permeability will result if the Hazen or Prugh methods are applied to these samples.
2. The samples used for particle size testing may be unrepresentative. When bulk or disturbed samples are recovered from below the water level in a borehole, there is a risk that finer particles will be washed from the sample. This is known as "loss of fines." Samples affected in this way will give overestimates of permeability if the Hazen or Prugh methods are used. Loss of fines is particularly prevalent in samples taken from the drilling tools during light cable percussion boring. This can be minimized by placing the whole contents of the tool (water and soil) into a tank or tray and allowing the fines to settle before decanting with clean water. Unfortunately, in practice, this is rarely done. Loss of fines is usually less severe for tube samples such as SPT or U100 samples; these methods may give more representative samples in fine sands.

6.7.5 Laboratory testing: Permeameter testing

There are a number of techniques for the direct determination of the permeability in the laboratory by inducing a flow of water through a soil sample—this approach is known as "permeameter" testing. According to Head (1982), there are two main types of permeameter testing.

1. Constant head test. A flow is induced through the sample at a constant head. By measuring the flow rate, cross-sectional area of flow, and induced head, the permeability can be calculated using Darcy's law. This method is only suitable for relatively permeable soils such as sands or gravels ($k > 1 \times 10^{-4}$ m/s); at lower permeabilities, the flow rate is difficult to measure accurately.
2. Falling head test. An excess head of water is applied to the sample and the rate at which the head dissipates into the sample is monitored. Permeability is determined from the test results in a similar way to a falling head test in a borehole or observation well. These tests are suitable for soils of lower permeability ($k < 1 \times 10^{-4}$ m/s) when the rate of fall in head is easily measurable.

Tests may be carried out in special permeameters, oedometer consolidation cells, Rowe consolidation cells, and triaxial cells. Methods of testing are described in BS 1377:1990 and in Head (1982).

Although these tests are theoretically valid, in practice, they are rarely used because of the difficulty of obtaining representative "undisturbed" samples of the granular soils (silt, sand, or gravel) of interest in dewatering design. Even if the sample is representative of the particle size distribution of the soil, the in situ density and hence void ratio of the soil is likely to be known only sketchily. This means that the in situ condition of the soil cannot be reproduced. Similarly, any soil fabric or layering in the sample will have a profound effect on the in situ permeability but cannot be replicated in the laboratory.

The only time such tests should be considered is when the permeability of very low-permeability soils needs to be determined during investigations of potential consolidation settlements. Even then, results must be interpreted with care, because in clays with permeable fabric, the size of the test sample may result in scale effects distorting the measured permeability; see the work of Rowe (1972). Large (250-mm-diameter) samples may give more representative results compared with the 76-mm-diameter samples routinely tested, but such large samples are rarely available.

6.7.6 In situ tests in boreholes and observation wells

A range of techniques are available to allow permeability to be derived from tests carried out either in boreholes (during pauses in the drilling process) or in observation wells installed in boreholes after the completion of drilling. There are a variety of methods and techniques which can be used, but most of them are based on a common principle—if water is added to or removed from a borehole in a controlled manner, observation of the resulting changes in water level in the borehole can be used to estimate the permeability of the surrounding ground.

Common forms of in situ permeability tests in boreholes include

1. Rising, falling, and constant head tests in boreholes (Section 6.7.7)
2. In situ tests in boreholes in rocks (Section 6.7.8)
3. In situ tests in observation wells (Section 6.7.9)

Because these tests add or remove relatively small quantities of water to or from the borehole, they can only influence the soil or rock locally around the borehole. Therefore these tests can, at best, produce "small scale" values of permeability representative of conditions around the borehole. Such tests may be unduly influenced by any effects of soil disturbance caused by drilling of the borehole or by local variations in geology close to the borehole. In contrast to in situ tests in boreholes, pumping tests (Section 6.7.11) typically influence a much larger volume of soil or rock and give more representative "large-scale" permeability values but are more time consuming and expensive to carry out.

The test methods described in the following sections are those which are applicable to conventional civil engineering and groundwater control projects. In recent decades, more sophisticated methods of permeability testing have been developed in related fields, including shaft sinking for deep mining (Daw 1984), investigations for deep geological disposal of nuclear waste (Sutton 1996), and carbon sequestration and storage (Wiese et al. 2010). These methods are not addressed here but use exactly the same principles as the tests described here, carried out at greater depths, at higher background pore water pressures, and in lower permeability rocks than is common for conventional tests.

6.7.7 Rising, falling, and constant head tests in boreholes

This group of tests includes

1. Rising and falling head tests (collectively known as variable head tests)
2. Constant head tests

These tests are carried out in the field on the soil in situ. They therefore avoid the problems of obtaining representative undisturbed samples that limit the usefulness of laboratory testing. Tests in boreholes are those carried out during pauses in the drilling or boring process. When the test is complete, drilling recommences—this allows several tests at different depths to be carried out in one borehole. These tests are distinct from tests carried out in observation wells (Section 6.7.9) after the completion of the borehole, where tests can be carried out only at the fixed level of the response zone.

Execution of variable head tests is straightforward and requires only basic equipment. The borehole is advanced to the proposed depth of the test, and the original groundwater level is noted. It is essential that a representative groundwater level is obtained. If necessary, the start of the test should be delayed until readings show that the pretest groundwater level has stabilized.

The upper portion of the borehole is supported by a temporary casing (which should be sealed into the upper strata to exclude groundwater from those levels). The "test section" of exposed soil is between the bottom of the casing and the base of the borehole.

For a falling head (or inflow) test (Figure 6.8a) water is rapidly added to increase the water level in the borehole. Once the water has been added, the water level in the borehole is recorded regularly to see how the level falls with time as water flows out of the borehole into the soil. The necessary equipment includes a dipmeter, bucket, stopwatch, and a supply of clean water (perhaps from a tank or bowser). It is essential that any water added is absolutely clean, otherwise any suspended solids in the water will clog the base of the borehole test section and significantly affect results. Particular attention should be given to the cleanliness of tanks and buckets so that the water does not become contaminated by those means. It can be difficult to carry out falling head tests in very permeable soils (greater than about 10^{-3} m/s) because water cannot be added quickly enough to raise the water level in the borehole. If the natural groundwater level is close to ground surface, it may be necessary to extend the borehole casing above ground level to allow water to be added.

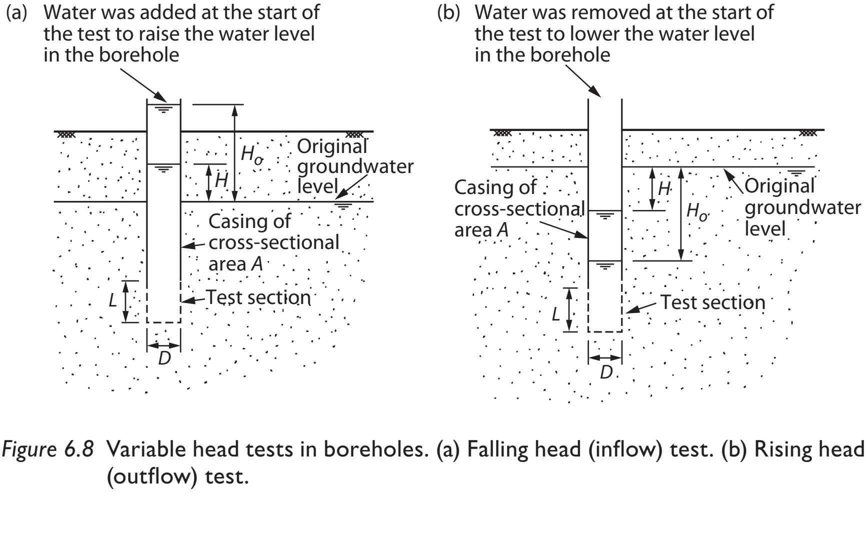


Figure 6.8 Variable head tests in boreholes. (a) Falling head (inflow) test. (b) Rising head (outflow) test.

A rising head (or outflow) test (Figure 6.8b) is the converse of a falling head test. It involves rapidly removing water from the borehole and observing the rate at which water rises in the borehole. The test does not need a water supply (which can be an advantage in remote locations) but does require a means of removing water rapidly from the borehole. The most obvious way to do this is using a bailer, which is adequate in soils of moderate permeability but it can be surprisingly difficult to significantly lower water levels if soils are highly permeable. Alternatives are to use airlift equipment or suction or submersible pumps.

An alternate form of variable head test is the "slug test." Again, this involves applying rapid changes to the water level in a borehole and then observing the rate at which the water level returns to the background or natural water level. However, in a slug test, no water is added to or removed from the borehole. Instead, a heavy rod (termed a slug) is quickly lowered below water level in the borehole to displace water and hence rapidly raise water levels (analogous to a falling head test). At the end of the falling head stage, when water levels have equilibrated, rapid removal of the slug from the water level will cause a sudden lowering of water level (analogous to a rising head test). Slug tests have the advantage that no water supply or equipment to pump or bail water is needed.

For the relatively permeable soils of interest in groundwater lowering problems, variable head tests can be analyzed using the work of Hvorslev (1951), which is the basis of the methods given in BS 5930:1999, amended 2010. Hvorslev assumed that the effect of soil compressibility on the permeability of soil was negligible during the test, and this is a tolerable assumption for most water-bearing soils. If in situ permeability tests are carried out in relatively compressible silts and clays, different test procedures and analyses may be required; see the work of Brand and Premchitt (1982).

For the Hvorslev analysis, permeability k is calculated using

$$k = \frac{A}{FT} \tag{6.3}$$

where A is the cross-sectional area of the borehole casing (at the water levels during the test), T is the basic time lag, and F is a shape factor dependent on the geometry of the test section. T is determined graphically from a semilogarithmic plot of H/H_o versus elapsed time as shown in Figure 6.9. H_o is the excess head in the borehole at time $t = 0$ and H is the head at time t (both H and H_o are measured relative to the original groundwater level). Additional notes on the analysis of variable head tests are given in Appendix 2.

Values of shape factor F for commonly occurring borehole test section geometries were prepared by Hvorslev (1951) and are shown in Figure 6.10. Shape factors for other geometries are given in BS 5930:1999, amended 2010. The simplest test section is when the temporary casing is flush with the base of the borehole, allowing water to enter or leave the borehole

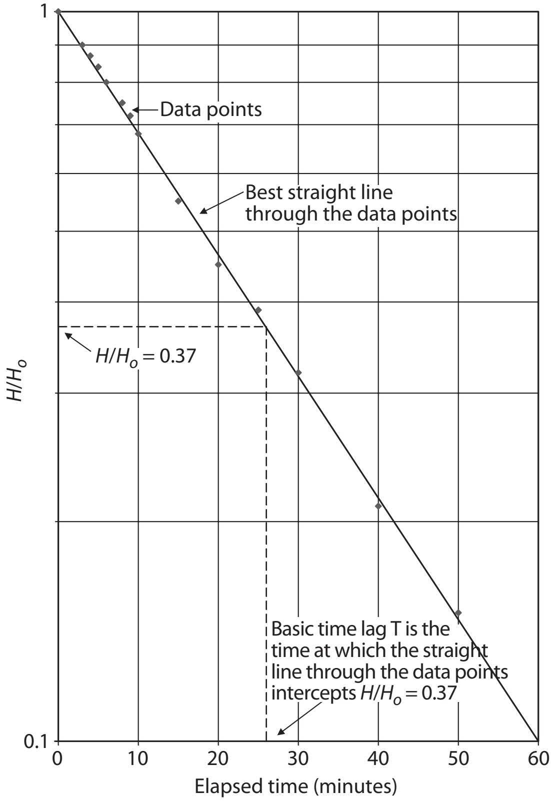


Figure 6.9 Analysis of variable head tests.

through the base only. If soil will stand unsupported, it may be possible to extend the borehole ahead of the casing to provide a longer test section. If the soil is not stable, the borehole could be advanced to the test depth, and the test section could be backfilled with filter sand or gravel as the casing is withdrawn to the top of the test section.

Constant head tests (Figure 6.11) involve adding or removing water from a borehole at a known rate to maintain a constant head, which is recorded. Constant head tests are most often carried out as inflow tests, but outflow tests can also be carried out. The equipment required is rather more complex than for variable head tests, as some form of flow measurement (typically by the timed volumetric method) is required. In the simplest form of the test, appropriate to relatively permeable soils, the flow rate is adjusted until a suitable constant head is achieved, and the test is allowed to continue until a steady flow rate is established. A consistent supply of clean water is required for tests, and this can be a disadvantage in remote locations.

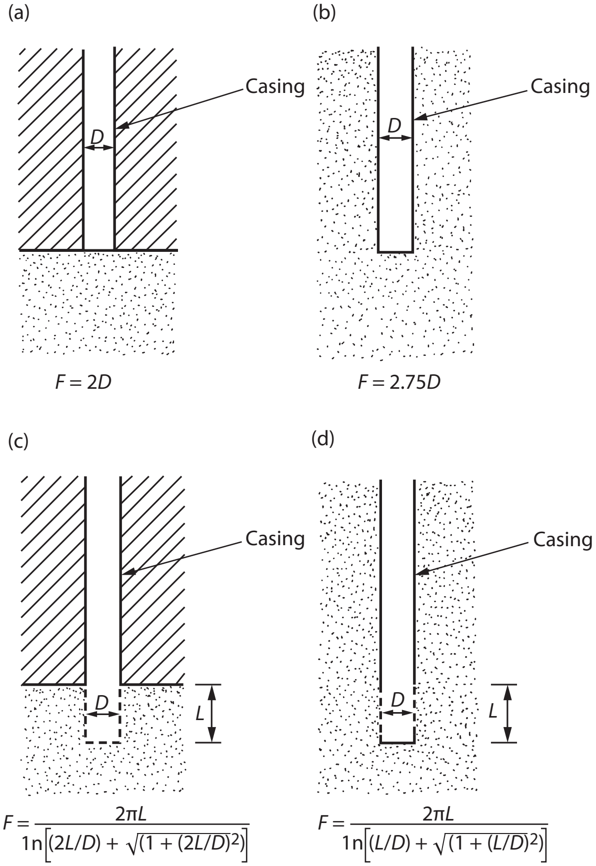


Figure 6.10 Shape factors for permeability tests in boreholes. (a) Soil flush with the bottom of the casing at the impermeable boundary. (b) Soil flush with the bottom of the casing in uniform soil. (c) Open section of the borehole that extended beyond the casing at the impermeable boundary. (d) Open section of the borehole that extended beyond the casing in uniform soil. (After Hvorslev, M.J., *Time Lag and Soil Permeability in Groundwater Observations*. Waterways Experimental Station, Corps of Engineers, Bulletin No. 36, Vicksburg, MS, 1951.).

Permeability k is calculated from

$$k = \frac{q}{FH_c} \tag{6.4}$$

where q is the constant rate of flow, H_c is the constant head (measured relative to original groundwater level), and F is the shape factor (from Figure 6.10).

It is well known (for example, see the work of Black 2010) that variable and constant head tests in boreholes have a number of limitations and may

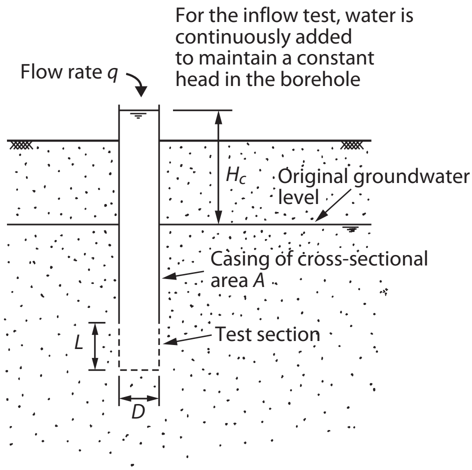


Figure 6.11 Constant head inflow test in boreholes.

be subject to a number of errors. When carrying out these tests (and when reviewing the results), it is essential that these factors are considered

1. Tests in boreholes only involve a relatively small volume of soil around the test section. If the soil is heterogeneous or has significant fabric, such tests may not be representative of the mass permeability of the soil. Large-scale tests (such as pumping tests) may give better results.
2. The conventional Hvorslev method of analysis is based on the assumption that the initial increase/decrease in water level in the borehole occurs instantaneously. Obviously, in practice, the change in water level will not be instantaneous but will require a finite period. All possible steps should be taken to keep to a minimum the period of adding/removing water at the start of the test. If this is not done, for example, where a hose pipe is used to continuously add water to a well over several minutes, the water level response will be different from the Hvorslev assumptions, and erroneous permeability values may result.
3. Results of inflow tests (falling head and constant head tests) can be significantly affected by clogging or silting up of the test section as water is added. It is vital that only totally clean water is added but, even then, silt already in suspension may block flow out of the borehole. It is not uncommon for inflow tests to underestimate permeability by several orders of magnitude.
4. In loose granular soils, outflow tests (rising head and constant head tests) may cause piping or boiling of soil at the base of the borehole. This could lead to overestimates of permeability.

5. The drilling of the borehole may have disturbed the soil in the test section, changing the permeability. Potential effects include particle loosening, compaction, or smearing of silt and clay layers.
6. Reliable analysis of test results requires that the original groundwater level be known (this is discussed in Appendix 2). A key issue is that where tests are carried out during pauses in drilling, it is likely that the drilling process will have affected groundwater levels. It is necessary to wait until monitoring has shown that groundwater levels have stabilized before commencing the test.
7. If the natural groundwater level varies during the test (because of tidal or other influences) the test may be difficult to analyze. If significant groundwater level fluctuations are anticipated during a test of, for example, 1- or 2-h duration, tests in boreholes are unlikely to be useful.
8. If the drilling casing does not provide an effective seal to isolate the test section, then leakage of water into or out of the test section may occur from other strata. This will affect the water level response during the test and may lead to erroneous results.

Although these tests have a number of limitations, they are inexpensive to execute and are widely used. It is good practice to carry out both rising and falling head tests in the same borehole to allow results to be compared. In any event, results from in situ tests in boreholes should be reviewed against the anticipated conceptual model (Section 7.4) for the site and treated with caution until supported by permeability estimates from other sources.

6.7.8 In situ tests in boreholes in rock

The borehole testing techniques used in soil can also be applied to boreholes in rock. However, in practice, a different approach is often taken to the in situ testing of boreholes drilled through rock strata. This is because the flow of groundwater will be mostly along joints, fissures, or other discontinuities. A borehole drilled through a stratum of rock may pass through relatively unfissured zones (which will be of low-permeability) and through more fissured zones (of higher permeability). It is important that the level and extent of these zones are identified. Two of the most useful approaches are geophysical logging of boreholes and packer permeability testing.

Geophysical formation logging of unlined boreholes in rock can help identify the presence of more or less fissured zones. Fluid logging methods (including flowmeter and fluid conductivity and temperature logging) can be used to determine specific levels at which groundwater is entering the well; see the work of Beesley (1986) and BS 7022:1988. The results from geophysical surveys can be used to specify the levels at which permeability testing should be carried out.

The packer test is one of the most common types of permeability tests used in boreholes drilled in rock, provided the borehole is stable without casing. The method is a form of constant head test, carried out within a discrete test section isolated from the rest of the borehole by inflatable "packers" (Figure 6.12). Water is pumped into or out of the test section and the change in water pressure or level noted. Because discrete sections of borehole at various depths can be tested, the method can help identify any fissured permeable zones. The packer test was originally developed in the 1930s to assess the permeability of grouted rock beneath dam foundations and is sometimes known as a Lugeon test, after the French engineer who pioneered the method. Strictly speaking, a true Lugeon test is one particular form of packer test, carried out using specific equipment and injection pressures, and the term should not be used for packer tests in general.

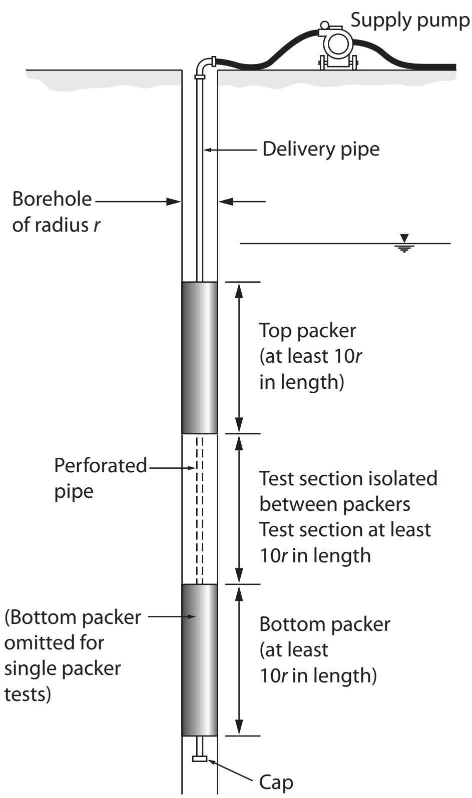


Figure 6.12 Packer test. (From Preene, M., Roberts, T.O.L., Powrie, W., and Dyer, M.R., Groundwater control—Design and practice, *CIRIA Report C515*. Construction Industry Research and Information Association, London, 2000. Reproduced by kind permission of CIRIA: www.ciria.org.)

The test method is described in Walthall (1990) and BS 5930:1999, amended 2010. In the double packer test, inflatable packers are used to isolate a test section between two packers (Figure 6.12). For a single packer test, the test section is between a packer and the base of the borehole. The most common type of packer test is an inflow test, in which water is injected into the test section and the flow rate and head recorded. Outflow tests can also be carried out, although the test equipment is more complicated (Price and Williams 1993); nevertheless, a number of studies (including Brassington and Walthall 1985) have concluded that outflow tests are preferable to inflow tests.

Packer tests are most suited to rocks of moderate to low-permeability. If permeability is greater than 10^{-7} m/s, friction losses in the pipework become significant and need to be included in the calculations. When packer tests are carried out in highly permeable zones (greater than about 10^{-5} m/s), it is difficult to inject sufficient water to maintain the test pressure, and the test may have to be aborted. Analysis of packer tests is described in Clayton et al. (1995) and BS 5930:1999, amended 2010. Tests can be carried out at various depths in an unlined borehole and may allow permeability depth profiles to be obtained. The following factors must be considered when carrying out packer tests (and when reviewing test results)

1. In most rocks, the overall permeability is dominated by flow via fissures. Measured permeability values will be affected if the drilling process has blocked or enlarged natural fissures. Walthall (1990) states that, before a packer test, the borehole should be cleaned out to remove all drilling debris and also recommends that the borehole be developed by airlifting. Even without drilling-related effects, the mere presence of the borehole may lead to stress relief and stress redistribution around the borehole, changing the local permeability.
2. As with any inflow test, it is vital that the injected water is absolutely clean so that the risk of siltation of the test section is reduced.
3. Care must be taken to ensure that injection pressures are not so high as to cause hydraulic fracturing or uplift of the ground. Even without hydraulic fracturing, high water pressures may artificially dilate existing fissures.
4. Problems sometimes occur if packers do not form an effective seal with the borehole walls. This will lead to leakage from the test section and can give completely misleading results. It is good practice to select the test sections based on the drilling records of the test hole and to try and locate packers in sections that are likely to give good seating for packers. If obtaining high-quality test results is important, it may be worthwhile to perform a caliper survey before packer testing and selecting packer setting depths on that basis.

6.7.9 In situ tests in observation wells

Variable and constant head tests can be carried out in observation wells (standpipes and standpipe piezometers) after the completion of boring (Figure 6.13). Although these tests can only be carried out at the depth of the observation well response zone, they have the advantage that they can be executed (and repeated if necessary) after boring has been completed; without the time pressures associated with working in pauses in the drilling process. Tests are analyzed using the same methods as for tests in boreholes. For observation wells with cylindrical response zones, the shape factor F derived by Dunn and Razouki (1975) can be used

$$F = \frac{2.32\pi D(L/D)}{\ln\left[1.1(L/D) + \sqrt{\left(1 + (1.1L/D)^2\right)}\right]} \tag{6.5}$$

where L is the length of the borehole test section and D is the diameter of the borehole. Where L is large in relation to D, the test will tend to determine the horizontal permeability k_h. In any event, if the horizontal permeability is much greater than the vertical, these tests will tend to measure k_h, whatever the ratio of L to D.

The limitations on variable and constant head tests in boreholes also apply to tests in observation wells. Additional problems may result from the nature of the observation well itself. If the standpipe or piezometer has not been installed to the highest standards, it may be partially clogged. In such cases, any tests will merely determine the permeability of the piezometer rather than the permeability of the soil. To reduce the risk of this problem, all observation wells should be purged or developed (by pumping or airlifting) before testing. Testing should not commence until the water level has recovered to its equilibrium level.

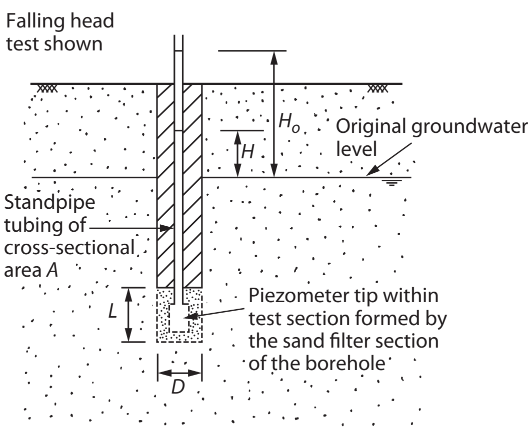


Figure 6.13 Variable head tests in observation wells.

6.7.10 Specialist in situ tests

Specialist in situ tests are occasionally carried out as part of certain types of probing techniques. These include dissipation tests carried out in piezocone testing (Lunne et al. 1997) or in situ permeameters (Chandler et al. 1990). These tests are generally used only in fine-grained soils (such as silt and clays) of low or very low permeability and are not generally used in investigations for groundwater lowering, except to investigate the permeability of aquitards, in which the risk of settlement is of concern.

The most common of these specialist tests is the dissipation test, carried out as part of piezocone probing. When the piezocone is advanced during probing in low-permeability soils, an excess of pore water pressure will build up at the cone, greater than the natural groundwater pressure, as a result of penetration. If penetration is stopped, the excess pore water pressure will dissipate at a rate controlled by the coefficient of consolidation with horizontal drainage c_h, which is related to the stiffness E'_o and horizontal permeability k_h of the soil by

$$c_h = \frac{k_h E'_o}{\gamma_w} \tag{6.6}$$

where γ_w is the unit weight of water. Analysis of the test involves determining the degree of dissipation U at a given time t, where

$$U = \frac{u_t - u_o}{u_i - u_o} \tag{6.7}$$

and

u_t = pore water pressure at time t
u_o = equilibrium pore water pressure at the level of the test
u_i = pore water pressure recorded at the start of the test

In general, the dissipation test should be continued at least until U is 0.50 or less. A typical dissipation test plot is shown in Figure 6.14. To estimate c_h, the time t_{50} for 0.5 degree of dissipation is determined and applied to the following formula:

$$c_h = \frac{T_{50}}{t_{50}} r_o^2 \tag{6.8}$$

where the time factor T_{50} is obtained from published theoretical solutions and r_o is the equivalent penetrometer radius. The theoretical solution to be used to estimate c_h should be chosen with care; it will be influenced by the strength characteristics of the soil and by the location of the pore water

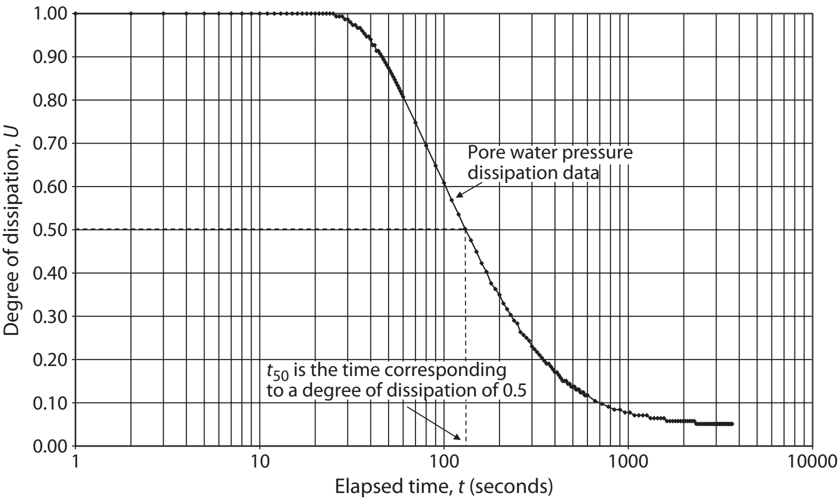


Figure 6.14 Analysis of pore water pressure data from piezocone dissipation test.

pressure measuring point on the piezocone tip; Lunne et al. (1997) present a number of possible solutions that may be used to obtain T_{50} and r_o.

Once c_h has been determined, k_h can be estimated using Equation 6.6, provided the soil stiffness is known. Determination of permeability from piezocone dissipation tests involves a lot of assumptions and uncertainties; values determined in this way should be treated as a general indicator only. The penetration of the piezocone itself may disturb the soil and may locally reduce the permeability around the cone. This can be particularly acute if the soil has a laminated structure, when the penetration of the cone through clay layers may smear clay over the more permeable sand layers. These factors may result in piezocone results that underestimate the in situ permeability.

6.7.11 Pumping tests

The simplest form of pumping test involves controlled pumping from a well and monitoring the flow rate from the well and the drawdown in observation wells at varying radial distances.

A correctly planned, executed, and analyzed pumping test is often the most reliable method of determining the mass permeability of water-bearing soils. This is principally because the volume of soil through which flow of water is induced is significantly greater than in the cases of variable and constant head tests in boreholes and observation wells. Unfortunately, because of the relatively high cost of a pumping test compared to these methods, it is used less frequently than is desirable.

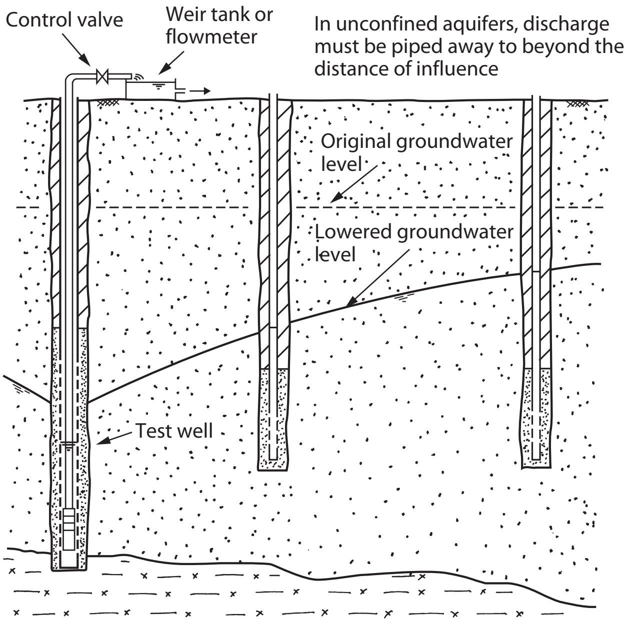


Figure 6.15 Components of a pumping test. For clarity, only two observation wells are shown, but several are normally required.

The essentials required for a pumping test (Figure 6.15) are

1. A central water abstraction point, usually a single deep well—although it can be a ring of wellpoints or ejectors—which forms the "test well." Ideally, this should penetrate the full thickness of the aquifer as this simplifies the analysis of test results.
2. A series of observation wells (standpipes or standpipe piezometers depending on the aquifer type) installed in the aquifer at varying radial distances from the test well. These allow the depth to groundwater level to be measured (manually by dipmeter or using datalogging equipment) to determine the drawdown at varying times after commencement of pumping and during recovery after cessation of pumping.
3. A means to determine the rate of pumping, such as a weir tank or flowmeter (see Section 16.4).

A pumping test typically consists of the following phases:

1. Pre-pumping monitoring
2. Equipment test
3. Step–drawdown test

4. Constant rate pumping phase
5. Recovery phase

In addition to monitoring of groundwater levels and the discharge flow rate, a test is also a useful opportunity to obtain groundwater samples for chemical testing. Further details on the execution of pumping tests is given in Appendix 3. Test methods are defined in BS 5930:1999, amended 2010 and BS ISO 14686:2003. The test well itself may also provide additional information about ground conditions. Geophysical logging of the well may provide useful information; see the work of Beesley (1986) and BS 7022:1988. In fissured rock aquifers, geophysical logging can allow the identification of fissured zones where groundwater is entering the well.

A pumping test is a relatively expensive way of determining permeability. The cost of a pumping test is seldom justified for a small project or for routine shallow excavations. However, for any large project or deep excavation, or where groundwater lowering is likely to have a major effect on the construction cost, one or more pumping tests should be carried out. It is essential that the pumping test is planned to provide suitable data for dewatering design. Issues to be considered include the drawdown during the test (which in nearby observation wells should be at least 10% of that in the proposed dewatering system), the number and position of observation wells (which should allow the drawdown pattern around the test well to be fully identified) and the design of the test well (which, ideally, should be of a similar design to and should be installed by the same methods as, the proposed dewatering wells). To meet these aims, a pumping test is generally carried out in the second or subsequent phases of ground investigation, when ground conditions have been determined to some degree. This allows the depth of the test well screen and observation well response zones to be selected on the basis of the data already gathered.

Analysis of the test results can provide information that is useful in a number of ways

1. Data from the step–drawdown test can be used to analyze the hydraulic performance of the well, to determine well losses and efficiency. This approach is widely used in the testing of water supply wells (see the work of Clark 1977 for methods of analysis) but is less widely relevant to temporary works wells for groundwater lowering purposes. It can still be useful when trying to optimize the performance of deep well systems and may allow a comparison of the performance of wells drilled and developed by different methods.
2. Data from the constant rate pumping phase can be used to estimate the permeability and storage coefficient of the volume of aquifer influenced by the test. The permeability estimated from the test results is used as an input parameter in the design methods described in Chapter 7. Permeability is estimated by conventional hydrogeological

analyses (described below), which can also provide some information about the aquifer boundary conditions.

3. Observations of the way drawdown reduces with distance from the test well can be used to construct a distance–drawdown plot, which can then be used to design groundwater lowering systems by the cumulative drawdown method (see Section 7.7). This method is interesting because the design does not need a permeability value because that is implicit in the distance–drawdown plot.

Most pumping tests are analyzed by "nonsteady state" techniques which are relatively flexible methods and can be applied to data even as the test is continuing. This allows data to be analyzed almost in "real time." "Steady state" methods of analysis can be used but may require much longer periods of pumping than is necessary with nonsteady state methods. In general, analysis by nonsteady state techniques is to be preferred.

There are a wide variety of methods of analysis which can be used to analyze the results of the constant rate pumping phase, many of which are usefully summarized in Kruseman and De Ridder (1990). Each of the methods is based on a particular set of assumptions about the aquifer system (unconfined, confined, or leaky), the well (fully or partially penetrating), and the discharge flow rate (generally assumed to be constant). Methods suitable for analysis of data from the recovery phase are also available.

These methods should be viewed as a "tool kit" providing a range of possible analysis methods. Provided that the basic details of the aquifer and well are known, it is normally straightforward to select one (or more than one, if uncertainty exists over aquifer conditions) method appropriate to the case in hand. Commonly used methods of analysis fall into two main types:

1. *Curve-fitting methods*. These typically involve plotting on a log–log graph, for each observation well, drawdown against elapsed time. The data will generally form a characteristic shape. The data curve is then overlain with a theoretical "type curve" and the relative positions of the two curves adjusted until the best match of the shape of the two curves is obtained. Once a match is achieved, permeability and storage coefficients can be determined by comparing values from each curve. These methods were developed from the work of Theis on simple confined aquifers, but variations are available for various other cases (see the work of Kruseman and De Ridder 1990). The curve-fitting process can be tedious but, in recent years, computer programs for pumping test analyses have speeded up the process.
2. *Straight-line methods*. This approach involves plotting sets of data so that characteristic straight lines are produced, allowing permeability and storage coefficients to be determined from the slope and position

> of the line. These methods are a special case of the Theis solution and are based on the work of Cooper and Jacob (1946) and are often called the Cooper–Jacob methods. Two approaches are possible and can be used on the same test. Time–drawdown diagrams involve plotting the drawdown data from one observation well against elapsed time since pumping began (Figure 6.16); this process is repeated for all observation wells being analyzed. Distance–drawdown diagrams plot the drawdown recorded (at a specific elapsed time) in all observation wells against the distance of each observation well from the test well (Figure 6.17).

The Cooper–Jacob straight-line method is the most commonly used method of analysis, mainly due to its relative simplicity. The original Cooper–Jacob method was based on horizontal flow to fully penetrating wells in confined aquifers but can also be used in unconfined aquifers where the drawdown is a small proportion (<20%) of the original aquifer-saturated thickness.

For the time–drawdown data from a single observation well, aquifer permeability k and storage coefficient S are determined as follows. From the semilog graph (Figure 6.16), draw a straight line through the main portion of the data (the data will deviate from the straight line at small times, and possibly at later times). From the graph, obtain the slope of the straight line, expressed as Δs, which is the change in drawdown s per log cycle of time.

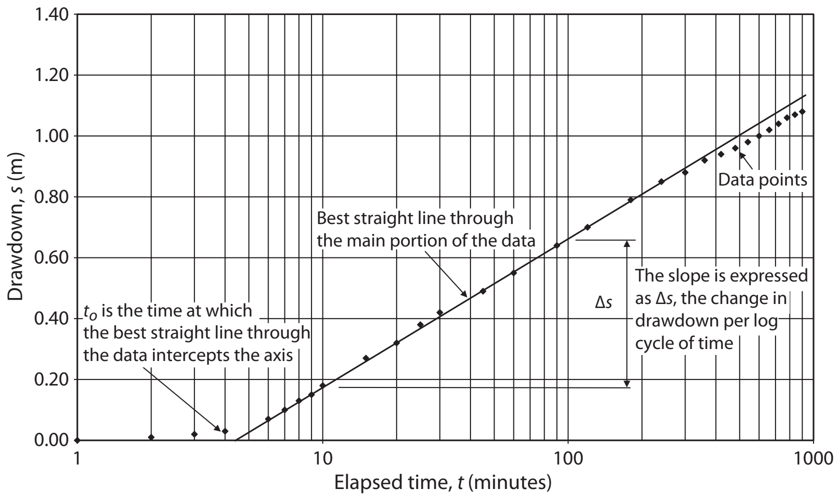


Figure 6.16 Analysis of pumping test data: Cooper–Jacob method for time–drawdown data.

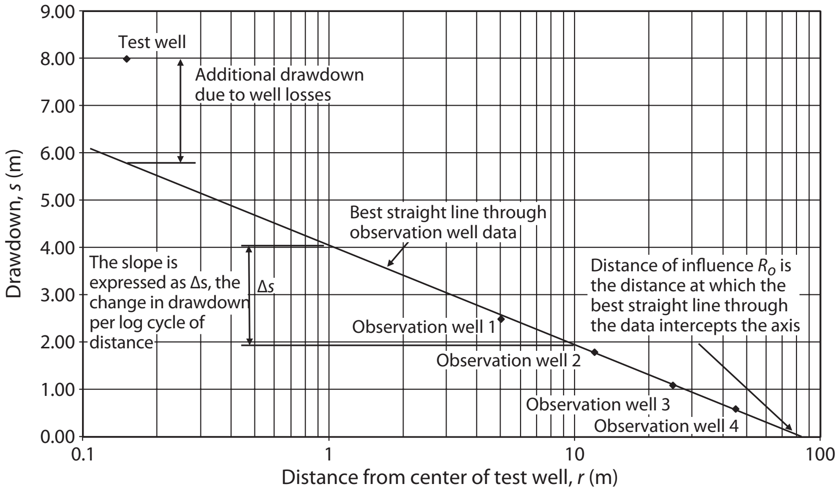


Figure 6.17 Analysis of pumping test data: Cooper–Jacob method for distance–drawdown data.

Also, determine t_0, the time at which the straight-line intercepts the zero drawdown line—*k* and *S* are then obtained from

$$k = \frac{2.3q}{4\pi \Delta s D} \tag{6.9}$$

$$S = \frac{2.25kDt_o}{r^2} \tag{6.10}$$

where *q* is the constant flow rate from the test well, *D* is the aquifer thickness, and *r* is the distance from the center of the test well to the observation well. This process can be repeated for each observation well.

For the distance–drawdown approach, data from all observation wells, at elapsed time *t* after pumping started, are plotted on the semilog graph (Figure 6.17). A straight line is drawn through the observation well data. If the test well has a much larger drawdown than the observation wells, this may be the result of well losses. In such cases, the straight line should be based on the observation wells only and should not include the test well drawdown. From the graph, obtain the slope of the straight line, expressed as Δs, which is the change in drawdown *s* per log cycle of distance. Also, determine R_o, the distance at which the straight-line intercepts the zero drawdown line—*k* and *S* are then obtained from

$$k = \frac{2.3q}{2\pi \Delta s D} \tag{6.11}$$

$$S = \frac{2.25kDt}{R_o^2} \tag{6.12}$$

The analysis can be repeated for various elapsed times. This approach also allows the distance of influence R_o to be estimated, which can be a useful check on values used in later dewatering designs.

Care must be taken to ensure that consistent units are used in these equations. To obtain permeability k in meters per second (its usual form in dewatering calculations), the following conventions are used: Δs, D, R_o, and r must be in meters; q must be in cubic meters per second (not liters per second—this is a common cause of numerical errors); and t and t_o must be in seconds (not in minutes, which is often the most convenient way to plot drawdown–time data).

Because the Cooper–Jacob method is only a special case of the more generic Theis solution, a check must be made to ensure that the method is valid. The Cooper–Jacob method can be used without significant error provided $(r^2S)/(4kDt) \leq 0.05$. This means that the approach is valid provided t is sufficiently large and r is sufficiently small.

Comprehensive analysis of a pumping test may produce several values of permeability. Time–drawdown analysis will produce one result for each observation well, and distance–drawdown analysis can produce several values, depending on how many time periods are analyzed. These permeability values are all likely to be slightly different, either due to uncertainties in analysis or in response to changes in aquifer conditions across the zone affected by the test. This highlights the fact that pumping test results may still need detailed interpretation; in complex cases, it is prudent to obtain specialist advice.

Variations on the Cooper–Jacob method for certain other aquifer conditions are given in Kruseman and De Ridder (1990).

6.7.12 Groundwater control trials

Groundwater control trials are an extension of the ethos of pumping tests, in that they are a large-scale in situ test to determine the hydraulic properties of the ground. In addition, a carefully planned trial can provide other useful information.

Instead of a single test well, a groundwater control trial involves pumping from a line or ring of wells of some sort (wellpoints, deep wells, or ejectors). Obviously, to specify a suitable trial, the main dewatering design must be reasonably advanced to allow selection of the appropriate pumping method, well spacing, well screen depth, etc. As with a pumping test, the trial is pumped continuously for a suitable period (typically 1–4 weeks) while monitoring discharge flow rate and water levels in observation wells. The results of such tests are normally analyzed using the design methods in

Chapter 7 to "back-calculate" an equivalent soil permeability. Powrie and Roberts (1990) describe an example of a trial using ejector wells.

In addition to determining the permeability, trials allow opportunities to investigate other issues relevant to the proposed works:

1. Because several wells have to be installed, the relative performance of proposed installation methods (e.g., drilling or jetting) can be assessed.
2. If the trial consists of a ring of wells, a trial excavation can be made inside the ring during the trial. This excavation can provide data on ease of excavation of the soil, stability of dewatered excavations, and trafficability of the plant across the dewatered soil. There have been cases in which trial excavations have been combined with large-scale compaction tests to assess the suitability of the excavated material as backfill elsewhere on site.

Figure 6.18 shows examples of groundwater control trials, in which rings of wellpoints were installed and the area within the ring excavated during pumping. This type of trial can not only provide information on the feasibility of dewatering at the site but also allow battered side slopes to be cut at various trial slopes and provide useful information on the handling characteristics of the dewatered soil.

Groundwater control trials are probably the most expensive form of in situ permeability test, but for large projects in difficult ground conditions, they can reduce the risk of problems during the main works, and the cost may be justified on that basis. If a dewatering system is to be designed by the observational method (see Section 7.3), a trial can be a key part of the scheme.

6.7.13 Comparison of methods

The foregoing sections have described the methods commonly used to estimate permeability. Meaningful and representative values of permeability are essential for the design of groundwater lowering projects, but the designer may be presented with a wide range of values taken by different methods in the same aquifer. Some of the variations in the reported values of permeability may result from a heterogeneity of the aquifer. Unfortunately, limitations in the test methods and the underlying assumptions in the analysis of test results may also introduce some "apparent" variation (independent of the properties of the aquifer) in the results.

Permeability is a difficult parameter to determine accurately and, when selecting permeability values to be used in design, some uncertainty is unavoidable. During the design process, this can be addressed by reviewing permeability values against the conceptual model and carrying out

Figure 6.18 Wellpoint dewatering trials. (a) Ring of wellpoints installed around trial area. A 20 × 20 m rectangular ring of wellpoints was installed around the trial area. Local excavations were made within the dewatered area to assess the behavior of the dewatered soil when excavated. (b) Battered excavation inside the wellpoint ring, with sides at various trial slope angles. A trial excavation was made within a 40 × 40 m rectangular ring of wellpoints in silty sand. Trial batter slopes were formed to assess the stability of the soil after groundwater lowering. (Courtesy of WJ Groundwater Limited, Bushey, U.K.)

Table 6.3 Tentative guide to the reliability of permeability estimates

Method	*Notes*	*Relative cost*	*Reliability*[a]
Visual assessment	For a soil or drift aquifer, can allow the order of magnitude of permeability to be estimated. Less applicable to fissured rock aquifers.	Very low	Good
Inverse numerical modeling	Only applicable if the hydrogeology of the aquifers being pumped is defined reasonably clearly, and if adequate monitoring data are available.	Low to moderate	Good
Geophysical methods	Can give indirect estimates of permeability and allow identification of zones of higher and lower permeability. Borehole geophysical methods are more applicable in rock aquifers to identify fissured horizons.	Moderate to high	Poor to good
Laboratory testing: particle size analysis	Can give reasonable results for samples of fairly uniform sand with low silt and clay contents, provided loss of fines has not occurred. Loss of fines tends to be greater from bulk samples compared with tube samples; permeability estimated from bulk samples should be treated with caution. Gives very poor estimates in laminated or structured soils or where silt and clay content is significant.	Very low	Very poor to good
Laboratory testing: permeameter testing	Of little use in granular sand or gravels, as representative samples cannot be obtained without unacceptable disturbance. Can produce good results in clays and some silts in which minimally disturbed samples can obtained. In soils with fabric and structure, permeability estimates are affected by sample size; smaller samples may underestimate permeability.	Low to moderate	Good
In situ tests in boreholes: falling head	Can test only a small volume of soil at the base of the borehole. May be influenced by soil disturbance due to boring. Prone to clogging of test section.	Very low	Very poor
In situ tests in boreholes: rising head	Can test only a small volume of soil at the base of the borehole. May be influenced by soil disturbance due to boring. Prone to clogging or loosening of test section. Better results are often obtained in coarse soils with little silt content.	Very low	Poor to moderate

(*continued*)

Table 6.3 (*Continued*) Tentative guide to the reliability of permeability estimates

Method	*Notes*	*Relative cost*	*Reliability*[a]
In situ tests in boreholes: constant head	Can test only a small volume of soil at the base of the borehole. May be influenced by soil disturbance due to boring. Inflow tests prone to clogging of test section. Better results are often obtained in coarse soils with little silt content.	Low	Very poor to moderate
In situ tests in boreholes: packer tests	Normally carried out in unlined sections of boreholes in rock. Results may be dominated by the presence of fissures intercepting the test section. Inflow tests prone to clogging of test section. Outflow tests generally provide better results but require more complex test equipment.	Low to moderate	Poor to good
In situ tests in observation wells: falling head	Highly dependent on the design of the observation well and any soil disturbance during installation. Can test only a small volume of soil near the test section. Prone to clogging of test section.	Very low	Very poor
In situ tests in observation wells: rising head	Highly dependent on the design of the observation well and any soil disturbance during installation. Can test only a small volume of soil near the test section. Better results are often obtained in coarse soils with little silt content.	Very low	Poor to moderate
In situ tests in observation wells: constant head	Highly dependent on the design of the observation well and any soil disturbance during installation. Can test only a small volume of soil near the test section. Inflow tests prone to clogging of test section. Better results are often obtained in coarse soils with little silt content.	Low	Very poor to moderate
Specialist in situ tests	Appropriate for silts, clays, and some sands. Can test only a small volume of soil near the probe but can be used to profile permeability with depth. If the tests involve penetrometers, disturbance or smear of soil layering may occur, affecting results.	Moderate	Moderate to good

(*continued*)

Table 6.3 (*Continued*) Tentative guide to the reliability of permeability estimates

Method	*Notes*	*Relative cost*	*Reliability*[a]
Pumping tests	Appropriate for soils of moderate to high permeability such as sands or gravels. Rarely appropriate for clays and silts, unless pumped by ejectors and monitored by appropriate types of observation wells (such as pneumatic piezometers). Affects a large volume of soil and can provide good values of mass permeability. Results can be difficult to interpret if multiple aquifer systems are present beneath the site.	Moderate to high	Good to very good
Groundwater control trials	Appropriate for large-scale projects in difficult ground conditions or where the observational method is being used. Requires careful planning.	High to very high	Very good

[a] When used in soil or rock conditions suitable to the method and when analyzed appropriately.

sensitivity analyses (see Chapter 7) to evaluate the effect of differing permeability on drawdown and flow rate.

When presented with permeability values from a range of methods, deciding on the range of permeability to be used in design calculations is made easier if the relative reliability of each method is known; some guidance is given in Table 6.3.

Chapter 7

Design of groundwater lowering systems

7.1 INTRODUCTION

The philosophy and basic methods for the design of groundwater lowering systems are outlined in this chapter. The main emphasis of this chapter and, indeed, of most of this entire book is to deal with field-proven, simplistic but practical, methods applicable to many common situations while providing advice on the approach to more complex problems.

The uncertainty inherent in any ground engineering process requires a "questioning" or "testing" approach be adopted in design, where nothing is taken for granted. Sensitivity or parametric analyses can be used to assess how the design could cope with differing conditions. Alternatively, the observational method might be used to vary the design based on records collected during construction. Both approaches are outlined, and the vital importance of developing a realistic conceptual model is stressed. The important effect that geometry and geological structure has on design is also described.

The basic designers "tool kit"—the formulae and concepts used in routine designs—are presented and their applications discussed. Methods for the estimation of steady-state discharge flow rate, and for the selection of well yield and spacing are described in detail. Other design issues (such as time to achieve drawdown) are also outlined. The basic tenets of groundwater modeling are discussed in relation to more complex problems. Several simple design examples are presented in Appendix 4.

7.2 WHAT IS DESIGN?

In the context of dewatering, design is the process of developing a workable and economic solution to a groundwater problem that will affect (or is already affecting, if dewatering is applied as a remedial measure) an excavation. In some cases, design starts with a clearly defined and understood groundwater problem, and the main design objective is to develop an appropriate solution. In other situations, the nature of the groundwater problem may not be well-understood, and the initial stages of design may involve an investigation and definition of the problem before specific solutions can be developed.

In essence, there are three elements to dewatering design:

1. *Modeling*: To develop a potentially viable dewatering solution, it is necessary to formulate and record the various elements of the problem—in other words, to develop a model that allows the problem to be described and communicated. Every dewatering problem should have a *conceptual model* developed at an early stage (see Section 7.4). For some complex or important dewatering designs, numerical models may be developed as part of later analysis. Development of a conceptual model is the foundation of good dewatering design. If this stage is neglected, then the chances of successful outcomes are reduced significantly.
2. *Analysis*: This is the stage of design at which numerical calculations or analyses are carried out, for example, to estimate the required pump flow rate or to assess potential environmental effects. Analysis could be carried out using the conventional analytical equations described in this chapter, or by using numerical modeling. It is essential that the analysis methods used are appropriate to the conceptual model developed previously.
3. *Judgment*: Modeling and analysis alone are not sufficient to ensure the design of effective groundwater control systems. An element of judgment is required to ensure that the proposed design is realistic and practicable. Judgment cannot be learned from a book such as this, but is developed through accumulated experience. However, as a guide, judgment might involve comparing the proposed dewatering design with established empirical guidance, or with examples of similar dewatering systems in comparable ground conditions. Some specific examples of the use of judgment are given in the empirical checks applied to the design examples in Appendix 4.

Successful design will typically require all three elements to be applied in some combination. Where dewatering design produces results that are perceived to be unsuccessful, perhaps where the target drawdown is not achieved or where it becomes clear that excessive pumping capacity has been provided, it is often found that one of the elements of design has been neglected.

In the authors' experience, the design of groundwater control systems is typically not governed in detail by national or international design standards. This is unlike some forms of geotechnical engineering design (for example, related to the design of piles or earth-retaining structures) in which design requirements may be set out more formally in design standards. For example, although Eurocode 7 (BS EN 1997-1:2004) includes a section on dewatering, it limits its requirements to relatively general comments that the design should be based on the results of ground investigations, the objective of design should be to achieve stable excavations and that dewatering systems should be reliable and subject to monitoring. This approach of setting "high-level" requirements, without setting prescriptive limits on possible design methods, is common with design standards in some other parts of the world.

7.3 DESIGN APPROACH

There are two main philosophical views of the design of groundwater lowering or dewatering systems:

1. That the design process is essentially a seepage calculation problem that can be assessed using groundwater and hydrogeological theory. This approach implies that the major problem is estimating the discharge flow rate and that the selection and design of equipment is a secondary matter, carried out once the flow rate has been determined. We could call this the "theoretical" approach in which the analysis element of design (see Section 7.2) dominates.
2. That the design process must concentrate on selecting the appropriate type of well, well spacing, and pump size for ground conditions. Direct estimation of the flow rate is a less important issue (and indeed may never actually be calculated precisely). This can be thought of as the "empirical" approach using case history experience in which the judgment element of design (see Section 7.2) dominates.

The authors consider that both theoretical and empirical approaches have their advantages and disadvantages depending on the ground conditions and on the nature of the project.

For example, there are a number of cases that are sufficiently common that, once site investigation has confirmed there are no unusual complications, they can be designed purely empirically—almost by rule of thumb, based on the established capabilities of standard dewatering equipment. For example, shallow trench excavations in homogenous sand deposits of moderate permeability can almost always be dewatered by wellpoint systems with a spacing of 1–2 m between wellpoints. This has been practically proven over many decades. The empirical method is not applicable in more complex (or less clearly identified) geometries and ground conditions. If applied in such circumstances, it can lead to considerable difficulties.

The theoretical approach can be used in cases whether or not there is empirical experience of the case in hand. The approach may involve fairly simple calculations or analyses, or may require more complex numerical modeling. The results of the calculations or modeling are used to specify the number and type of wells, pumps, etc., that will be required. Problems often arise when the theoretical approach does not take into account the limitations and advantages of the various dewatering techniques. If these issues are not considered, impractical or uneconomic system designs may result.

The best design approaches incorporate elements of both the theoretical and empirical methods. The theoretical method requires a "conceptual model" of the ground and groundwater regime to be developed, following which calculations are carried out. Simple and fairly basic calculations are perfectly acceptable and may be preferred in many cases, provided they are compared with an empirical approach. The empirical method should be used as a "sanity check" to ensure that the proposed groundwater lowering system is realistic and practicable. For example, if the output of a theoretical design recommends a single stage of wellpoints to achieve a drawdown of 8 m, this is clearly not going to work. More subtly, if a wellpoint spacing of, say, 15 m was recommended, this should be looked into more closely because this is outside the normal range of wellpoint spacings—there may be problems with the conceptual model, methods of analysis, or selection of dewatering method.

This chapter presents the methods that can be used in combined theoretical and empirical approaches. The main emphasis is to deal with field-proven practical means pertinent to most groundwater lowering projects. The various methods presented form a tool kit of techniques that, if selected with care, can deal with a wide range of actual problems.

One defining feature of the design of any geotechnical process (including groundwater lowering) is that there will be some uncertainties in the ground. These uncertainties may result from the site investigation being of limited or inappropriate scope. Alternatively, even after a comprehensive site investigation, the sheer variability and complexity of the revealed ground conditions may give rise to uncertainty in design.

Uncertainty will affect the way the design progresses. In principle, there are two basic approaches to design:

1. Predefined designs. This might be thought of as the "traditional" design process, whereby one geological profile and set of parameters is selected and used to produce a single set of design predictions. The design is implemented with fairly basic monitoring, limited to checking that the design is "effective" in the gross sense. Design and construction methods are not reviewed unless the original design is "ineffective" (e.g., not achieving the target drawdown in a dewatering design). In such cases, corrective action (alternative design and construction methods) would be taken.
2. The observational method. This contrasting design process begins with more than one geological profile and set of parameters and more than one option for the required dewatering system. These might

range from "most probable" to "most unfavorable," perhaps with a number of intervening conditions as well. More than one set of design predictions are produced, together with measurable "trigger values" to allow the detailed effectiveness of the design to be observed during construction. The data extracted during construction are continually reviewed, to allow design and construction methods to be altered incrementally to match the behavior of the ground and groundwater.

7.3.1 Predefined designs for groundwater lowering

The predefined approach to dewatering design is to select a geological profile and important parameters (primarily permeability) and then apply these to an appropriate method of seepage analysis (such as one of those presented in Section 7.7). The result of this calculation is the "estimated" or "design" discharge flow rate and drawdown distribution. Such an approach sometimes has an air of certainty or finality, especially if carried out by civil engineers more used to relatively consistent and reliable materials like steel and concrete.

In reality, few successful groundwater lowering systems are designed in such a regimented way. As was discussed in Chapter 3, permeability and aquifer boundaries can have a dramatic effect on dewatering systems; the difficulty of accurately determining permeability was described in Section 6.7. These factors mean that it is unrealistic to carry out a single seepage analysis and to then expect the dewatering system to have a high likelihood of performing adequately.

The best groundwater lowering systems are flexible and robust in nature, able to cope with ground conditions slightly different from those anticipated with few, if any, minor modifications. Such systems are also easy to modify or upgrade if ground conditions are substantially different from those expected.

The predefined approach can still be used but, normally, more than one set of calculations is carried out as part of "sensitivity" or "parametric" analyses.

A sensitivity analysis is a set of repeated calculations, using varying values of a single parameter in each calculation. Typically, for dewatering designs, permeability is a key parameter, and thus, seepage calculations are carried out for a range of possible values. The question being addressed is: Can the groundwater lowering system, as designed, cope with the range of discharge flow rates corresponding to the possible range of permeabilities? If the answer is yes, all is well. If the answer is no, but the system could be easily (and quickly) modified to handle flow rates at either extreme end of the range, then the system may still be acceptably robust. If the answer is no and the system cannot be readily modified, there is a risk that the system will not be able to handle all the possible flow rates. In such cases, it may be prudent to try and develop an alternative, more flexible design, and test that against the results of the sensitivity analysis.

Parametric analyses are a rather broader version of sensitivity calculations. The question addressed is: What parameters are influential to the design? A number of parameters or conditions are systematically varied in calculations to investigate which have the greatest effect on the design. This may allow the design to be varied, or may prompt additional site investigation work to refine estimates of certain parameters or clarify key issues. In addition to permeability, aquifer boundary conditions can have an important effect on dewatering designs. A parametric study might look at the effect of a close source of recharge affecting the distance of influence, or the depth of the aquifer being deeper than expected. Again, the design process should include an assessment of the ability of the proposed dewatering system to cope with such possible conditions.

7.3.2 Observational method for groundwater lowering

The observational method allows a rational approach to construction where there is uncertainty over ground conditions or over the most suitable and economic dewatering or ground treatment options. The method uses construction observations to gather information about the behavior of the ground and groundwater and then modifies the methods in response. The method is much more than carrying out parametric or sensitivity analyses. It should be a continuous and deliberately planned and managed process of design, construction control, monitoring, and review that allows previously defined modifications to be incorporated into the construction process when necessary. The object is to achieve a robust process that provides for economic construction without compromising safety. The background, methods, and case histories of the observational method are described in CIRIA Report C185 (Nicholson et al. 1999). Two main variants are normally identified:

1. *Ab initio*—applied from the inception of the project.
2. *Best way out*—applied during construction to allow progress when unexpected problems occur on-site. It is often applied when a "predefined" design has proved unsatisfactory, and modifications are required.

The observational method can be applied to many groundwater lowering systems. This is because they can be easily modified (by the addition of extra wells or by using pumps of different capacity) and because easily observable parameters (such as drawdown and discharge flow rate) can be used to interpret how the system is performing. Examples of the observational method applied to groundwater lowering systems are given in the work of Roberts and Preene (1994b), Nicholson et al. (1999), and Preene et al. (2000).

The best way out application of the observational method is when a predefined method does not work and an alternative must be developed. It has been used when dewatering systems have had to be uprated or modified when the predefined design fails to achieve the design aims. Effectively, the initial system (which was not installed with the observational method in mind) is monitored and used as a large-scale pumping test or trial to allow remedial measures to be selected.

The *ab initio* approach has been widely applied to larger projects, where ground conditions are known to be complex, or where the project design is not going to be finalized until construction is well advanced. The number of wells and pumping capacity can be optimized based on the data gathered during the groundwater lowering works themselves. Optimization of the systems should be carefully considered, to avoid the temptation of using the bare minimum number of wells. A truly optimized system will have an adequate standby plant, alarm systems, plus some additional wells (over and above the minimum) as an allowance against loss of wells or performance due to construction damage, clogging, or biofouling and so on.

One possible reason why the *ab initio* application of the observational method has mainly been applied to larger projects is the need for clear management of the design and construction process. This is necessary to allow construction feedback to be obtained and linked into the ongoing design process in a timely manner. If smaller contracts are managed on this basis, there is no reason why the benefits of the observational method could not be applied to a wider range of projects.

The methods presented in the remainder of this chapter are based largely on predefined methods of design, but applied to give robust and flexible systems. The observational method will not be considered further, but it is an approach that the dewatering designer should be familiar with because it is another part of the tool kit to be applied when appropriate.

7.4 DEVELOPMENT OF A CONCEPTUAL MODEL

The essential first step in the design process is the development of a conceptual model of the ground and groundwater conditions. A conceptual model is described by Brassington and Younger (2010) as "a theory-based description that represents the phenomena being studied that is founded on a set of variables with logical and quantitative relationships."

Until the conceptual model is developed, it is not possible to make rational decisions about which of the methods in the designer's tool kit is suitable for the case in hand. To develop a conceptual model, the designer needs to be familiar with the concepts of groundwater flow, aquifers, boundary conditions, and so on (see Chapter 3).

If a conceptual model is a poor match for actual conditions, then much of the subsequent design work may be of dubious value. Even if the design work is diligently and carefully carried out, if it is based on an inappropriate conceptual model, the design may be going in the totally wrong direction—and this is likely to lead to poorly performing dewatering systems. Developing the conceptual model at an early stage also forces the designer to review the available data. If the designer has insufficient reliable data to formulate a model, this could be a sign that further site investigation is needed.

The conceptual groundwater model depends on a number of factors that are dictated by ground conditions and are beyond the designer's control. A very wide range of factors are relevant to hydrogeological conceptual models, as described by Brassington and Younger (2010). For the purposes of dewatering design, key factors to be incorporated in the dewatering design include

1. *Aquifer type(s) and properties.* Aquifers (the water-bearing strata in which groundwater levels are to be lowered) may be classified as confined or unconfined. These types behave in quite different ways; the aquifer type(s) must be identified. It is essential that the likely permeability range of each aquifer be identified. If transient analyses are to be carried out, the storage coefficient must also be estimated.
2. *Aquifer depth and thickness.* These dimensions must be estimated, and judgment should be made as to whether they are effectively constant across the area affected by the dewatering or whether the aquifer thickness varies significantly.
3. *Presence of aquitards and aquicludes.* Very low-permeability silt or clay layers may act as barriers to vertical groundwater flow. The presence of such strata may necessitate well screens above and below the aquiclude or aquitard.
4. *Distance of influence and aquifer boundaries.* Is all the pumped water likely to be derived purely from storage in the aquifer? If not, are there any nearby sources of recharge water? The presence of any barrier boundaries can also influence groundwater flow.
5. *Initial groundwater level and pore water pressure profile.* The initial groundwater level determines the amount of drawdown required. Complications may arise if the groundwater level slopes across the site, or if groundwater levels vary with tidal or seasonal influences.

6. *Presence of compressible strata*. If present in significant thickness, this indicates that potentially damaging consolidation settlements may occur.

There are also some factors that are not directly related to ground conditions. Some of these can be controlled by the designer:

1. *Geometry of the works*. The depth and size of the excavation will have a direct influence on the groundwater lowering requirements. The target lowered groundwater level is normally set a short distance (e.g., 0.5–1 m) below the excavation formation level. The drawdown needed is taken as the vertical distance from the original groundwater level to the target lowered groundwater level.
2. *Groundwater lowering technique*. Different dewatering methods may interact with the groundwater regime in quite different ways.
3. *Period for which groundwater lowering is required*. The time for which pumping is to continue may influence the choice of technique.
4. *Depth of wells*. In very deep aquifers, it may be more economic to install wells which do not penetrate to the base of the aquifer. Instead shallower, partially penetrating wells may be more suitable.
5. *Environmental constraints*. The location or nature of the site and its surroundings may give rise to limits on discharge rates or drawdown being imposed by the client or environmental regulator. These limits are normally imposed to reduce the risk of detrimental environmental effects from groundwater lowering (see Chapter 15). Occasionally, these constraints can severely limit the options available to the dewatering designer.

If all, or at least most, of these questions can be answered, a conceptual model can normally be developed. The conceptual model need not be particularly complex, and may simply be a list of the expected conditions. Figure 7.1 shows an example of a simple pro forma to record key data. Conceptual models are outlined in each of the design examples given in Section 7.11.

Any consideration of groundwater flow in general, or of the equations presented later in this chapter, highlight aquifer permeability as a critically important parameter. Chapter 6 has described the plethora of techniques available to estimate permeability, from the simple to the very complex. When assessing permeability values to be used in calculation, the designer should not visualize a single permeability value to be used in the conceptual model, but rather a range of realistic values to be used in sensitivity analyses. The range of permeability values may represent uncertainty due to natural variations in permeability, or limitations in the permeability test methods or results.

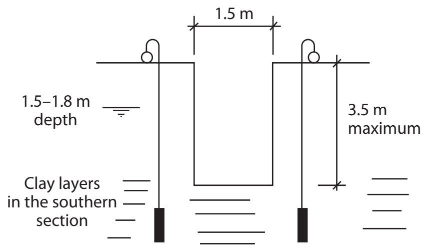
PMC DEWATERING	
Site:	Eastern resewerage, phase 1
Location:	Anytown
Prepared by:	MP
Basic information	
Aquifer type and properties:	Unconfined aquifer. Generally described as fine to medium sand. PSD results indicate *k* range of 1 to 5 × 10^{-4} m/s. Falling head tests give lower results.
Initial groundwater level:	1.5 mbgl at the northern end, varying steadily to 1.8 mbgl at the southern end. No long-term monitoring was carried out, so the background or seasonal variations are unknown.
Depth to base of aquifer:	Not proved. Deepest borehole penetrated to 18 m depth and was still in sand.
Aquitards/aquicludes present:	Some thin clay layers at 3–5 m depth in the southern section; could result in overbleed seepage.
Recharge/barrier boundary:	None apparent
Excavation depth and geometry:	Trench works, mainly 2.5–3.5 m deep. Trench width less than 1.5 m.
Maximum drawdown:	Maximum drawdown is 0.5 m below deepest dig = 4 m depth. This implies a maximum drawdown of 2.5 m.
Dewatering period:	4 weeks per 100-m section.
Compressible strata:	None indicated in site investigation data.
Possible dewatering technique:	Wellpoints are parallel to the trench. Wellpoints are 6 m deep and, therefore, will be partially penetrating. There is a risk of overbleed in the south.
Sketch:	
Other notes	

Figure 7.1 Example of a simple conceptual model for a groundwater lowering system.

It is difficult to provide simple, useful guidelines on selection of realistic permeability ranges. There will always be some reliance on judgment and experience, but the following advice is relevant:

- Be aware that different methods of assessing permeability produce results of greater or lesser reliability. Table 6.3 provides some guidance. Consider the relative merits of each method when assessing permeability from the available results.
- Always compare the permeability results with "typical" values of permeability from published correlations with soil types (such as Table 3.1) or, even better, from experience at nearby sites. This approach is vital in excluding unrealistically high or low-permeability results. For example, few experienced engineers would expect a slightly silty sandy gravel to have a permeability of 1×10^{-8} m/s, yet falling head tests in such soils often produce results of that order.
- If permeability estimates from various differing techniques produce broadly similar results, in agreement with typical values for those soil types, then the design range of permeability could be assessed from the full range of data. If there are large discrepancies in the data from various methods, some of the data may have to be excluded from the assessment process. Again, Table 6.3 may be of help in assessing the reliability of the data.
- Always consider the important aspects of the design when selecting the permeability range to be used in calculations. Mistakes have been made when designers have focused too much on estimating the highest likely permeability, to ensure a pumping capacity sufficient for the maximum possible flow rate. This is not always the most appropriate approach. It is true that in high-permeability aquifers, the total flow rate may be critical, and assessing permeability at the upper end of the possible range may be a robust approach. However, in soils of low to moderate permeability, a critical case in design may be if the permeability is at the lower end of the possible range, at which point yields may be very low, necessitating an unfeasibly large number of wells.

Once the conceptual model exists, an initial view must be taken of the dewatering method to be used and the likely geometry of the system.

7.5 EXPECTATIONS OF ACCURACY

It is important to be realistic with the expectations of accuracy. Dewatering design should not be viewed as a precise analytical process resulting in single numerical values for key design outcomes such as pumped flow rates. The preceding parts of this chapter and, indeed, much of the rest of this book have repeatedly highlighted the likely uncertainty in design parameters and boundary conditions, and the resulting need to consider a range of design scenarios using sensitivity and parametric studies. In most circumstances, a dewatering design should not report a single value for calculated quantities such as pumped flow rate, time to achieve drawdown, estimated ground settlement, or lowering of water levels at nearby groundwater-dependent features. Use of single values in design reports and calculation summaries may give the reader a false sense of precision; it is preferable in calculations and design reports that a range of values be quoted, for example "the predicted pumped flow rate will be between 10 L/s and 15 L/s, based on the assumptions stated in the calculations."

Figure 7.2 presents some interesting data from a study of around 20 pumped groundwater control systems in soils of low to moderate permeability, where dewatering was carried out by wellpoints, deep wells, or ejectors. For each case, the pumped flow rate (q_c) was predicted by standard analytical methods, based on a conceptual model and parameter values carefully derived from the available ground investigation information. These values were then compared with the pumped steady-state flow rate recorded in the field (q_r). This study showed that, even with high standards of conceptual modeling, design, and parameter selection, the best that could be achieved was to predict the pumped flow rate within a factor of three times greater than, or less than, the flow rate observed in the field (Figure 7.2). In a small number of cases shown in Figure 7.2, the observed pumped flow rate varied from the observed value by a factor greater than three. In these cases, retrospective analysis indicated the actual ground conditions differed significantly from those indicated in the site investigation information.

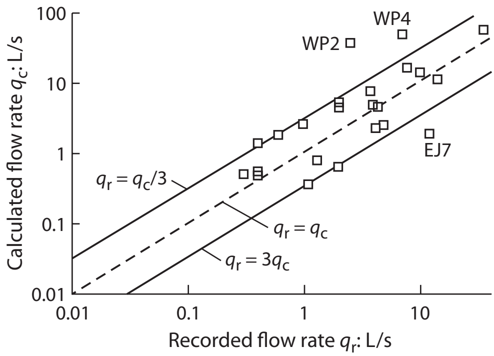


Figure 7.2 Comparison of calculated and recorded flow rates from dewatering systems. (From Preene, M., and Powrie, W., *Géotechnique*, 43, 2, 191–206, 1993: Reproduced by permission of ICE Publishing.)

The data in Figure 7.2 do not indicate that methodical and rational dewatering design is futile. Rather, they show that the designer should avoid assigning undue precision to calculated values. The design process should identify the most likely value for calculated values, but should also indicate the likely maximum and minimum values. These maximum and minimum values are important as they need to be compared with the capabilities of the proposed dewatering system. For example, if the proposed dewatering system can cope with the predicted maximum and minimum values of flow rate, either in its nominal layout or with easily applicable modifications, then a robust dewatering design is likely to result. However, if the maximum and minimum predictions could not be handled by the dewatering system, or would require expensive and time-consuming modifications, it may be appropriate to revisit the selection of the dewatering method and equipment and to develop a more flexible solution.

7.6 SELECTION OF METHOD AND GEOMETRY

There are a number of groundwater lowering methods available (see Section 5.5), and part of the design process is to select an appropriate technique which will satisfy the various constraints on the project in hand.

A useful starting point when selecting a technique is Figure 5.6. Knowing the required drawdown and estimated soil permeability from the conceptual model, the appropriate method can be chosen. Where more than one method is feasible, the choice between them may be made on the grounds of cost, local availability of equipment, or expertise of those carrying out the works.

7.6.1 Equivalent wells and slots

In practice, seldom can the required drawdown for an excavation be achieved by a single well; most dewatering systems rely on several wells acting in concert. The rare exceptions being, in a high-permeability stratum, the use of a large central sump pumping installation with radial drains (see Section 8.9 for a case history). The collector well system (see Section 11.6) is a variation on this concept, but is more generally used for agricultural or land drainage purposes than for civil engineering construction (although a case history is given in Section 14.7).

For dewatering purposes, the established principle is to install an array of water abstraction points, generally sited immediately adjacent to the area of the proposed excavation. This preferred location of wells (at the perimeter of the excavation) is based on practical experience. If wells are sited within the area to be excavated, they are greatly at risk because maintaining wellheads, riser pipes, power cables, and discharge mains is rarely compatible with the activities of a heavy excavation plant. Where possible, wells are best located immediately outside the excavation area or within the batter of excavations. Wells located in slope batters, although presenting an initial excavation and access impediment, are inherently more secure (in comparison with wells in the main excavation area) as work progresses deeper.

Very large or wide excavations often cannot be dealt with adequately or economically by wells sited only around the perimeter. As highlighted previously, the practicalities of maintaining and reinstalling damaged wells in the middle of a "live" area of excavation are far from trivial. In many such cases, unpumped relief wells (see Section 11.5) could be used inside the excavation area. These act as vertical drainage paths and their usefulness is not affected by progressive shortening as the excavation is deepened. The disadvantage of relief wells is their continuous water discharge; which must be disposed of by an effective drain and sump system. The disposal problem is readily controlled where the rates of flow are small (i.e., the underlying stratum is of low-permeability, say, less than 10^{-5} m/s).

In general, the multiple well systems used for groundwater lowering can be classified as either "linear" systems (for installation alongside trench excavations) or "ring" systems (for installation around circular or rectangular excavations). Figure 7.3 shows the definitions of these geometries.

This leads to the practical question of how we can model the flow to such systems. It would be very tedious to consider the flow to each individual well and the complex interactions between them. A useful approach is to consider the groups of wells as large "equivalent wells" or "equivalent slots," thereby allowing simple and accessible formulae to be used to estimate flow rate.

We can define an equivalent well as a groundwater lowering system where, on a gross scale, flow of groundwater to the system is radial. Radial flow implies that flow lines converge to the well from a distant diffuse source of water. The concept of the "zone of influence" of a well (introduced in Section 3.5) describes the area of the aquifer affected by pumping

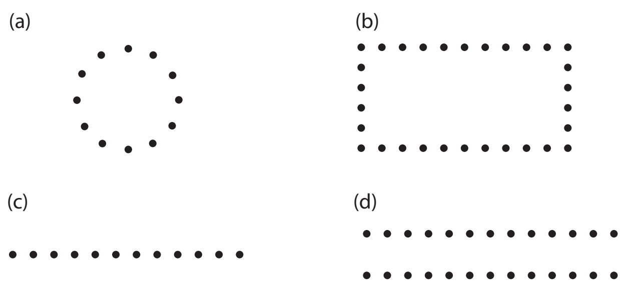


Figure 7.3 Plan layouts of groundwater control systems: linear and ring arrays. (a) Circular ring of wells. (b) Rectangular ring of wells. (c) Linear single line of wells. (d) Linear parallel line of wells.

from a well (Figure 7.4a). The radius of influence R_0 is a theoretical concept describing the zone of influence. R_0 is defined as the distance from the center of the well to the edge of an idealized circular zone of influence.

An equivalent slot is a line of closely spaced wells forming a groundwater lowering system. If the line of wells is very long, the flow of groundwater to the system is plane (although there will be some radial flow to the ends of the slot). Under plane conditions, flow lines do not converge, but are parallel, resulting in quite different flow conditions from radial flow. The edge of the theoretical zone of influence will be parallel to the line of wells (Figure 7.4b). The distance of influence, L_0, is the distance from the line of wells to the idealized edge of the zone of influence.

Of course, the equivalent well and slot concepts are approximations, introduced purely to make the estimation of discharge flow rate more amenable to simple solutions. Nevertheless, there is considerable justification for using these simplifications in appropriate conditions. The equivalent well concept was proposed by Forcheimer (1886). He based his work on the work of Dupuit and analyzed radial flow toward a group of wells. By correlation with field data, he demonstrated the acceptability of the concept for most practical purposes; provided that the wells are spaced in a regular pattern. The equivalent well concept was later endorsed by Weber (1928), based on extensive field data. The equivalent slot approach, in which a long line of closely spaced wells is treated as one continuous water abstraction slot, is implicit in the work of Chapman (1959) who studied flow to wellpoint systems. The equivalent well and slot simplification is an established practical method, used in the design sections in the work of Powers et al. (2007) and in CIRIA Report C515 (Preene et al. 2000).

For radial flow to rings of wells, this approach requires an estimation of the equivalent radius, r_e, of the system (Figure 7.5). For a circular ring of wells, r_e is simply the radius of the ring. For a rectangular ring of wells of

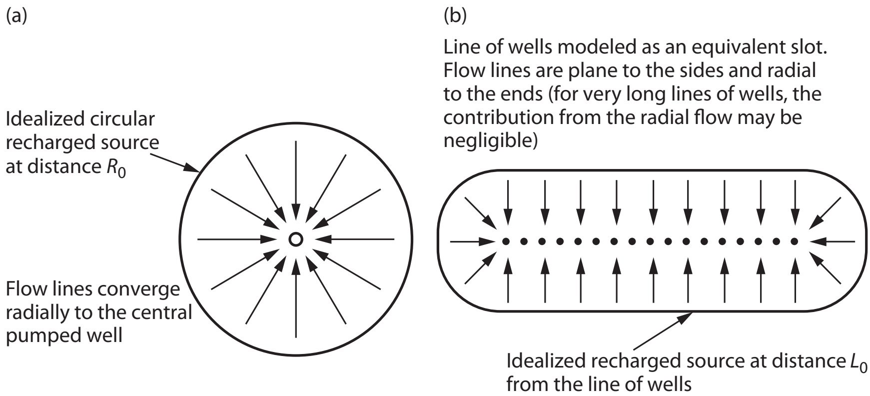


Figure 7.4 Zone of influence. (a) Radial flow. (b) Plane flow.

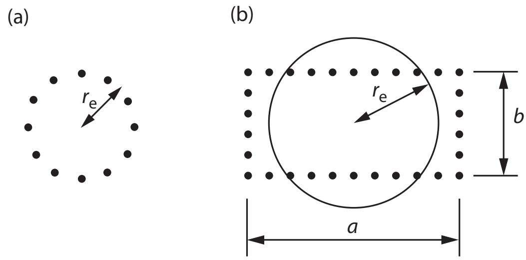


Figure 7.5 Equivalent radius of arrays of wells. (a) Circular system of radius r_e. (b) Rectangular system.

plan dimensions *a* by *b*, the equivalent radius can be estimated by assuming a well of equal perimeter

$$r_e = \frac{(a+b)}{\pi} \tag{7.1}$$

or equal area

$$r_e = \sqrt{\frac{ab}{\pi}} \tag{7.2}$$

In practice, both formulas give similar results, provided the ring of wells is not very long and narrow (i.e., provided *a* is not substantially greater than *b*). The estimate of r_e determined in this way can be used in the well flow equations presented in Section 7.7.

Long narrow systems consisting of lines of closely spaced wells (in which *a* is much greater than *b*), or in which the distance of influence is small, are likely to operate in conditions of plane flow (as opposed to radial flow). These systems may be better simplified to equivalent slots (Figure 7.6). In addition to plane flow to the sides, there will be a component of radial flow at the end of the line of wells (Figure 7.4). For relatively long systems, the radial flow component is likely to be relatively minor and is sometimes neglected. For shorter systems, the radial flow to the end may be a significant proportion of the total discharge and should be incorporated in calculations.

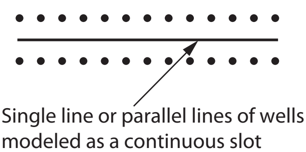


Figure 7.6 Equivalent slots.

7.6.2 Geological structure, well depth, and underdrainage

Geological layering and structure may have a controlling effect on the geometry of groundwater lowering systems, in particular, the well depth and the level of well screens. There are situations in which it may be possible to use the geological structure to advantage to enhance the performance of the dewatering system. The potential to do this should have been identified in the conceptual model. Some options are discussed in the following paragraphs.

In a homogenous permeable aquifer, the wells must penetrate to sufficient depth to achieve the required drawdown. As a rule of thumb, widely spaced wells should penetrate to one-and-a-half to two times the depth of the excavation, to ensure that the wells have adequate "wetted screen length" (see Section 7.8) even after drawdown. In an aquifer that extends to great depth, the wells may not need to penetrate to the base of the aquifer, but may be designed to be "partially penetrating" with a depth controlled by the need for adequate wetted screen length (Figure 7.7).

If the aquifer does not extend to great depth below the excavation formation level and is underlain by an aquiclude or aquitard, the wells will have to fully penetrate the aquifer. In practice, obtaining the required drawdown for excavation can be very problematic if the residual aquifer thickness below the excavation is much less than around one-third of the original saturated thickness (Figure 7.8). Obtaining the final part of the drawdown is difficult because the presence of the low-permeability layer restricts the wetted screen length of each well. This reduces the yield of each well and its corresponding ability to generate further drawdown. There are two possible approaches in this case. One is to install the wells at much closer

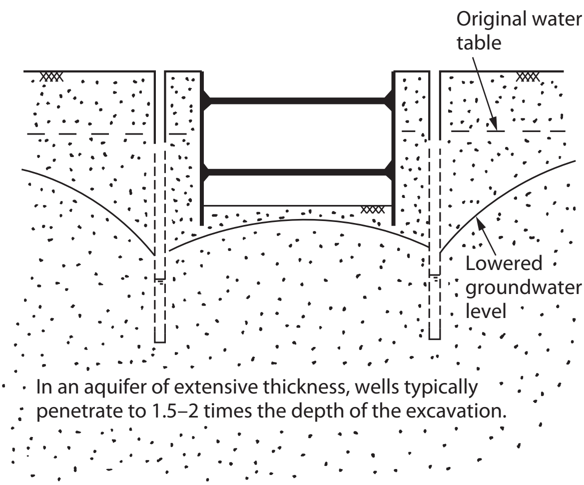
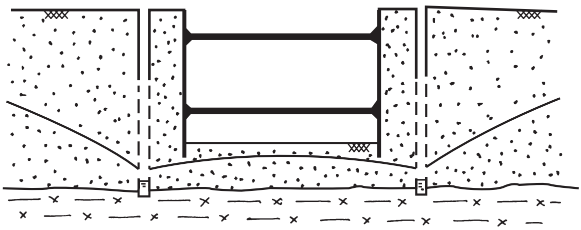


Figure 7.7 Wells in aquifers extending to a great depth.



The presence of a very low-permeability layer a short distance below the excavation formation level limits the potential wetted screen length of each well.

Figure 7.8 Wells in aquifers of limited thickness.

spacings than normal to obtain adequate wetted screen length by having a large number of wells. This approach can be economic with the wellpoint or ejector method, but less so with deep wells, due to the greater cost per well. The second approach is to install an economic number of wells and to then try and manage the residual "overbleed seepage" (see Section 4.7) by protecting the faces of the excavation with sandbags, geotextile filters, or other erosion prevention measures (Figure 9.13b). This allows water to enter the excavation without causing damaging loss of fines; the seepage water must then be removed by sump pumping.

If the aquifer is not homogenous, but consists of layers of greater and lesser permeability, well depth and screen level must be dictated by layering. The basic requirement is to abstract water *directly* from the most permeable layers (or more strictly, the layers of the highest transmissivity) in preference to intermediate or lower permeability layers. This will ensure that well yields are maximized because the permeable layers will readily feed water to the well screens.

If the most permeable layer (such as a gravel stratum) is beneath layers of moderate permeability (such as a silty sand), the most efficient system will involve wells pumping directly from the gravel. This will promote downward drainage of water from the sand to the gravel and thence to the wells (Figure 7.9). This important case is known as the "underdrainage" approach and is widely used where ground conditions allow. It is nearly always the best option, even if the wells have to be slightly deeper than first thought to intercept the permeable layer.

In contrast, where a permeable layer overlies a less permeable layer, it is likely that a dewatering system abstracting purely from the lower layer may not achieve the required groundwater lowering. The rate of recharge to the more permeable stratum will exceed the rate at which water can be abstracted from the underlying lower permeability layer. Hence, if an excavation has to penetrate into the underlying stratum of lesser permeability, it will be necessary to provide two abstraction systems. One system must pump

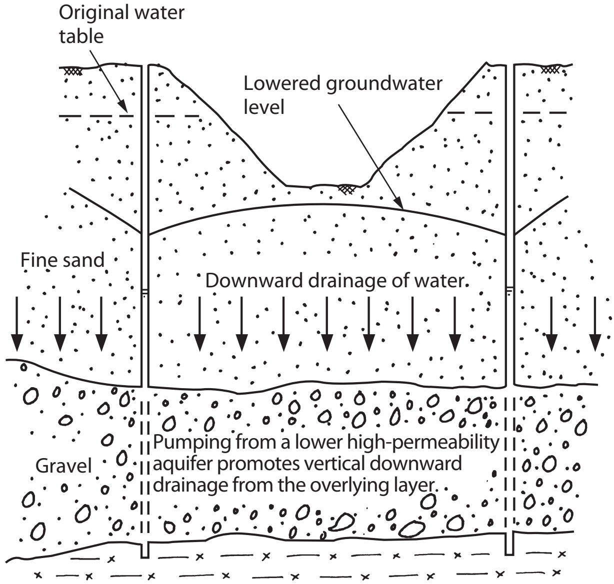


Figure 7.9 Pumping from a layered aquifer system using the underdrainage principle.

from the overlying more permeable layer. This allows the second pumping installation, screened in the underlying and less permeable stratum, to operate without being overwhelmed by seepage from the overlying layer.

7.7 ESTIMATION OF STEADY-STATE DISCHARGE FLOW RATE

Two key unknowns to be determined during design are the steady-state discharge flow rate and the yield, number, and design of wells necessary to achieve that flow rate. Some commonly used methods to estimate discharge flow rate are described in this section. Estimation of well yields is described in Section 7.8.

7.7.1 Steady-state well and slot formulae

This section presents simple formulae that can be used to estimate the steady-state discharge flow rate from systems treated as equivalent wells or slots. The emphasis here is on simple methods, to be used when the conceptual model indicates conditions that are not too different from the idealizations and assumptions discussed in the following paragraphs. For more complex cases, or where conditions differ dramatically from the simple conditions discussed here, analysis by numerical modeling may be appropriate (see Section 7.10).

The simple formulae for radial flow to wells are generally based on the work of Dupuit (1863), which used certain simplifications and assumptions about the aquifer's properties and geometry (Figure 7.4).

- The aquifer extends horizontally with uniform thickness in all directions without encountering intermediate recharge or barrier boundaries within the radius of influence.
- Darcy's law is valid everywhere in the aquifer.
- The aquifer is isotropic and homogeneous; thus, the permeability is the same at all locations and in all directions.
- Water is released from storage instantly when the head is reduced.
- The pumping well is frictionless and fully penetrates the aquifer.
- The pumping well is very small in diameter compared with the radius of influence, which is an infinite source of water forming a cylindrical boundary to the aquifer at distance R_0.

In reality, several of these assumptions are unlikely to be fully satisfied. For example, soils are usually stratified and generally exhibit horizontal permeabilities in excess of those in the vertical direction; often, by more than one order of magnitude. Similarly, in rock, the permeability may be dominated by fissure flow and may vary greatly from point to point.

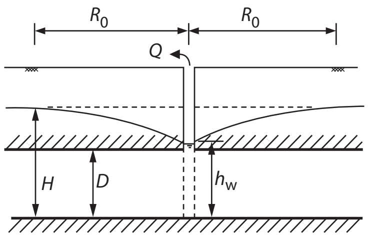
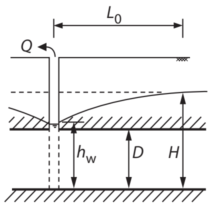
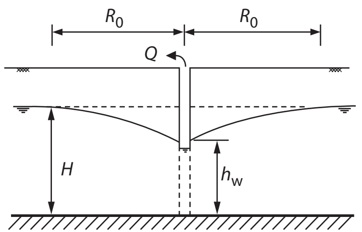
Dupuit made a further, important, assumption. This was that the groundwater flow to the well was horizontal. This is a valid assumption for fully penetrating wells in confined aquifers, but is invalid (at least close to the pumping well) in unconfined aquifers or if the well is only partially penetrating. Dupuit's analysis was purely for the radial flow case, but Muskat (1935) did analogous studies for plane flow to slots, using similar, idealized, assumptions.

Nevertheless, despite the idealizations and simplifications inherent in the formula, experience has demonstrated that the Dupuit-based formulae can be successfully used to estimate the steady-state pumping requirements for relatively short-term dewatering purposes. These methods are used in the design sections in the work of Mansur and Kaufman (1962) and Powers et al. (2007).

The empirical evidence that the Dupuit methods provide reasonable estimates of flow rate is supported by a number of theoretical studies. Hantush (1964) stated that "the Dupuit-Forcheimer well discharge formulae, despite the shortcomings of some of the assumptions, predict the well discharges within a high degree of accuracy commensurate with experimental errors." The assumptions have a more significant effect on the accuracy of the lowered groundwater level profile around a well, but even then, it is generally accepted that the Dupuit approach can predict drawdowns to acceptable accuracy at distances from the well of more than one-and-a-half times the aquifer thickness.

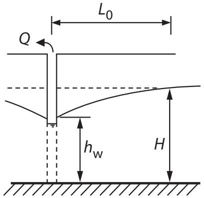
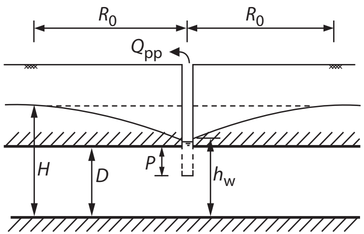
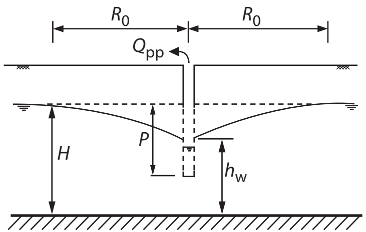
The commonly used formulae for estimation of the steady-state discharge flow rate are listed in Table 7.1, together with diagrams of the idealized

Table 7.1 Simple formulae for estimation of steady-state flow rate

Case	*Schematic diagram*	*Formula for steady-state flow rate, Q*		*Notes*
Radial flow to wells				
Fully penetrating well, confined aquifer, circular source at distance R_0 (Theim equation)		$Q = \dfrac{2\pi kD(H - h_w)}{\ln[R_0/r_e]}$	(7.3)	k = soil permeability; D = thickness of confined aquifer; H = initial piezometric level in aquifer; h_w = lowered water level in equivalent well; r_e = equivalent radius of well; R_0 = radius of influence.
Fully penetrating well, confined aquifer, line source at distance L_0 (method of images)		$Q = \dfrac{2\pi kD(H - h_w)}{\ln[2L_0/r_e]}$	(7.4)	k = soil permeability; D = thickness of confined aquifer; H = initial piezometric level in aquifer; h_w = lowered water level in equivalent well; r_e = equivalent radius of well; L_0 = distance to line source.
Fully penetrating well, unconfined aquifer, circular source at distance R_0 (Dupuit–Forcheimer equation)		$Q = \dfrac{\pi k(H^2 - h_w^2)}{\ln[R_0/r_e]}$	(7.5)	k = soil permeability; H = initial water table level in aquifer; h_w = lowered water level in equivalent well; r_e = equivalent radius of well; R_0 = radius of influence.

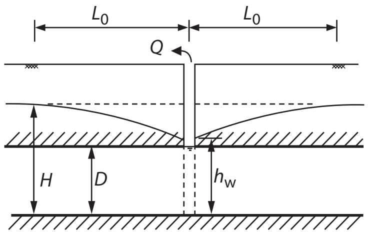
(*continued*)

Table 7.1 (*Continued*) Simple formulae for estimation of steady-state flow rate

Case	*Schematic diagram*	*Formula for steady-state flow rate, Q*		*Notes*
Fully penetrating well, unconfined aquifer, line source at distance L_0 (method of images)		$Q = \dfrac{\pi k\left(H^2 - h_w^2\right)}{\ln\left[2L_0/r_e\right]}$	(7.6)	k = soil permeability; H = initial water table level in the aquifer; h_w = lowered water level in an equivalent well; r_e = equivalent radius of the well; L_0 = distance to the line source.
Partially penetrating well, confined aquifer		$Q_{pp} = BQ_{fp}$	(7.7)	Q_{pp} = flow rate from a partially penetrating well; Q_{fp} = flow rate from a fully penetrating well; B = partial penetration factor for radial flow (obtained from Figure 7.10a). P = depth of penetration of well into aquifer
Partially penetrating well, unconfined aquifer		$Q_{pp} = BQ_{fp}$	(7.7)	Q_{pp} = flow rate from a partially penetrating well; Q_{fp} = flow rate from a fully penetrating well; B = partial penetration factor for radial flow (obtained from Figure 7.10b). P = depth of penetration of well below original water table.

Plane flow to slots

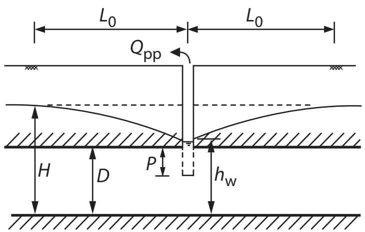
Fully penetrating slots, confined aquifer, and flow from line sources on both sides of the slot



$$Q = \frac{2kDx\left(H - h_w\right)}{L_0} \qquad (7.8)$$

x = linear length of the slot;
k = soil permeability;
D = thickness of the confined aquifer;
H = initial piezometric level in aquifer;
h_w = lowered water level in equivalent well;
L_0 = distance of influence.

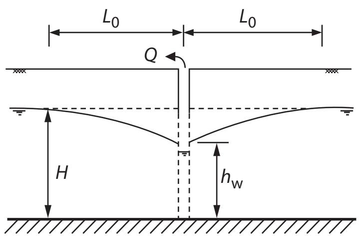
Partially penetrating slots, confined aquifer, and flow from line sources on both sides of the slot



$$Q_{pp} = \frac{2kDx\left(H - h_w\right)}{\left(L_0 + \lambda D\right)} \qquad (7.9)$$

x = linear length of slot;
k = soil permeability;
D = thickness of confined aquifer;
H = initial piezometric level in aquifer;
h_w = lowered water level in equivalent well;
L_0 = distance of influence;
P = depth of penetration of slot into aquifer
λ = partial penetration factor (obtained from Figure 7.10c).

Fully penetrating slots, unconfined aquifer, and flow from line sources on both sides of the slot

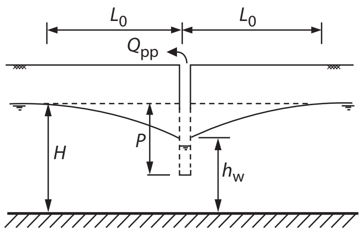


$$Q = \frac{kx\left(H^2 - h_w^2\right)}{L_0} \qquad (7.10)$$

x = linear length of slot;
k = soil permeability;
H = initial water table level in aquifer;
h_w = lowered water level in equivalent well;
L_0 = distance of influence.

(*continued*)

Table 7.1 (*Continued*) Simple formulae for estimation of steady-state flow rate

Case	*Schematic diagram*	*Formula for steady-state flow rate, Q*	*Notes*
Partially penetrating slots, unconfined aquifer, and flow from line sources on both sides of slot		$$Q = \left[0.73 + 0.23(P/H)\right]\frac{kx\left(H^2 - h_w^2\right)}{L_0} \quad (7.11)$$	x = linear length of slot; k = soil permeability; H = initial water table level in the aquifer; h_w = lowered water level in an equivalent well; L_0 = distance of influence; P = depth of penetration of the slot below the original water table.
Plane and radial flow			
Rectangular systems, confined aquifer		$$Q = kD\left(H - h_w\right)G \quad (7.12)$$	k = soil permeability; D = thickness of the confined aquifer; H = initial piezometric level in the aquifer; h_w = lowered water level in an equivalent well; G = geometry shape factor (obtained from Figure 7.11).

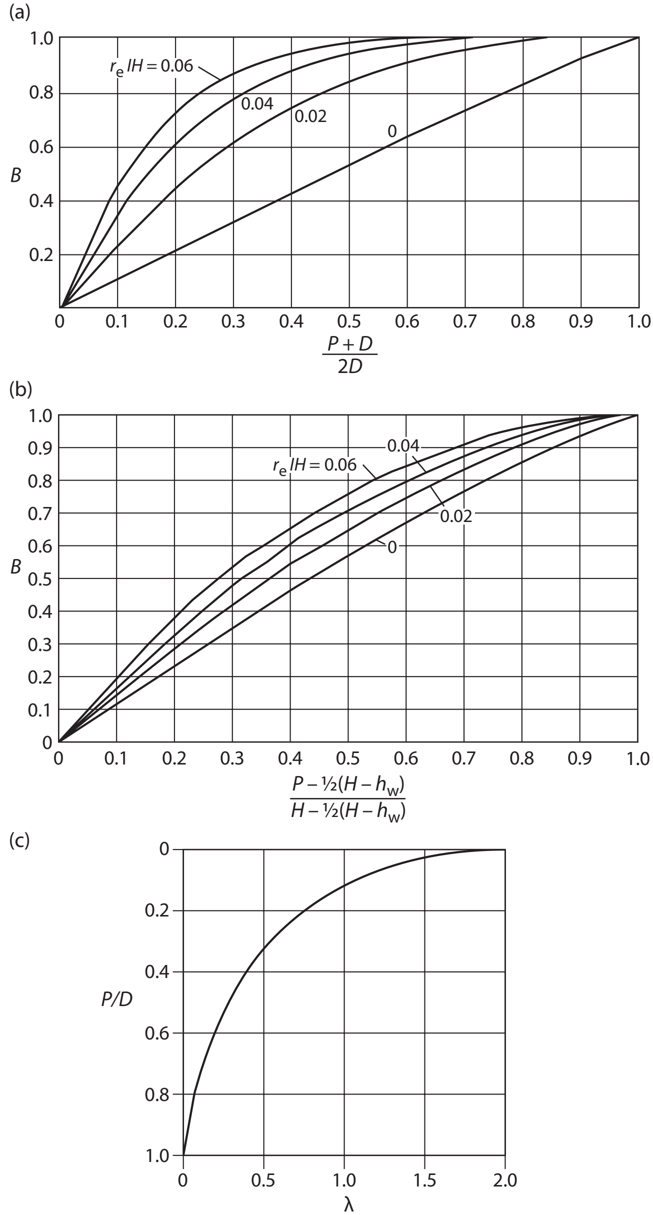


Figure 7.10 Partial penetration factors for wells and slots. (a) Radial flow to wells in confined aquifers. (b) Radial flow to wells in unconfined aquifers. (c) Plane flow to slots in confined aquifers. (After Mansur, C.I., and Kaufman, R.I., *Foundation Engineering* (edited by G.A. Leonards), McGraw-Hill, New York, 241–350, 1962.)

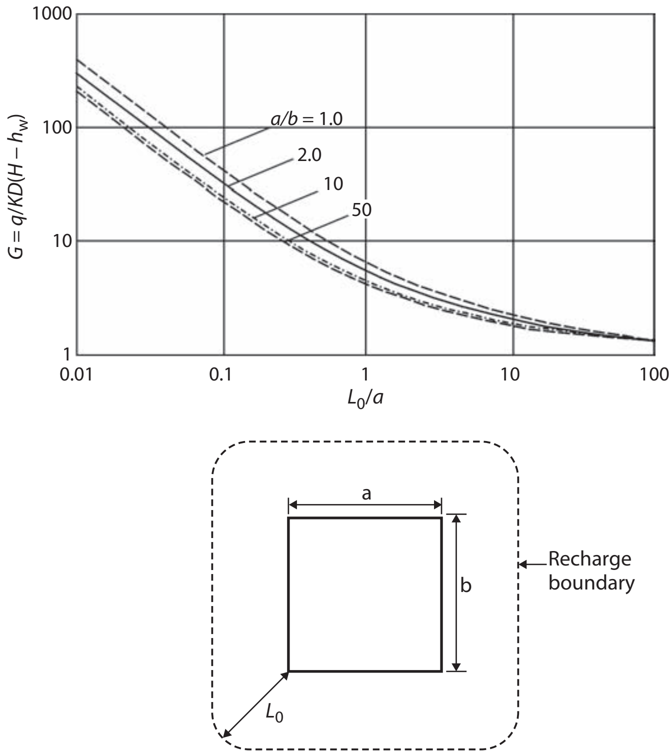


Figure 7.11 Shape factor for a confined flow to rectangular equivalent wells. (From Powrie, W., and Preene, M., *Géotechnique*, 42, 4, 635–639, 1992: Reproduced by permission of ICE Publishing.)

geometry. The formulae are categorized by whether the aquifer is confined or unconfined, whether flow is radial or plane, and whether the well or slot is fully or partially penetrating. All these conditions must be clarified during the development of the conceptual model before the formulae can be applied.

A significant qualification on the use of these formulae is that the results will only be as valid (or invalid!) as the parameters used in them. Previous sections have discussed the selection of permeability values for design purposes and the need for sensitivity and parametric analyses. A similar approach should be applied when using the steady-state formula. Additionally, other parameters should be selected with care, including

1. Equivalent radius (r_e) of system. For radial flow cases, this can be estimated from Equations 7.1 or 7.2.
2. Radius of influence (R_0) for radial flow cases. The radial flow cases assume a circular recharge boundary at radius R_0. This is a theoretical

concept representing the complex behavior of actual aquifers (see Section 3.5); the distance of influence is not a constant on a site, but is initially zero and increases with time. However, the simplification of an empirical R_0 value is a useful one. The most reliable way of determining R_0 is from pumping test analyses presented as a Cooper–Jacob straight line plot of distance–drawdown data (see Section 6.7). If no pumping test data are available, approximate values of R_0 (in meters) can be obtained from Sichardt's formula (which is actually based on earlier work by Weber)

$$R_0 = 3000\left(H - h_w\right)\sqrt{k} \tag{7.13}$$

where $(H - h_w)$ is the drawdown (in meters) and k is the soil permeability (in meters per second). This formula needs to be modified when used to analyze large equivalent wells. Dupuit assumed that the radius of the well was small in comparison to the radius of influence, but often the radius r_e may be large in comparison to R_0. In such cases, the following equation can be used.

$$R_0 = r_e + 3000\left(H - h_w\right)\sqrt{k} \tag{7.14}$$

When estimating R_0, it is important to review the calculated distance of influence to avoid using wildly unrealistic values. In the authors' experience, values of less than around 30 m or more than 5,000 m are rare and should be viewed with caution. It may be appropriate to carry out sensitivity analyses using a range of distance of influence values to see the effect on calculated flow rates. For the radial flow case, R_0 appears in a log term so small errors do not have a significant effect on calculated flow rates, but the possibility of gross error exists if a very large or very small R_0 is used.

3. Distance of influence (L_0) for plane flow cases. L_0 (in meters) can be estimated from Sichardt's formula, but a different calibration factor must be used.

$$L_0 = 1750\left(H - h_w\right)\sqrt{k} \tag{7.15}$$

where k is in meters per second and $(H - h_w)$ is in meters. The distance of influence appears as a linear term in the plane flow equations—the estimated flow rate is inversely proportional to L_0. The distance of influence must be chosen with care, and sensitivity analyses are strongly recommended.

4. Lowered water level (h_w) inside the equivalent well or slot. The equivalent well or slot method requires that the lowered water level (inside the well or the slot) used in equations is the groundwater level in the

excavation area itself. Obviously, the water level in each individual well will be lower (perhaps considerably so), but this drawdown would not be representative of the drawdown in the equivalent well or slot.

7.7.2 Cumulative drawdown analysis—Theoretical method

The formulae described in the previous section are used to analyze systems of closely spaced wells, modeled as equivalent wells or slots. Such an approach is less satisfactory if the wells are widely spaced; in those cases, a cumulative drawdown (or superposition) method may be more suitable.

This method takes advantage of the mathematical property of superposition applied to drawdowns in confined aquifers. In essence, the total (or cumulative) drawdown at a given point in the aquifer, resulting from the action of several pumped wells, is obtained by adding (or superimposing) the drawdown from each well taken individually (Figure 7.12). This approach is theoretically correct in confined aquifers, but is invalid in unconfined aquifers where the changes in saturated thickness that occur during drawdown complicate the interaction of drawdowns.

Expressed mathematically, the superposition principle means that the cumulative drawdown ($H - h$) at a given point as a result of n wells pumping from a confined aquifer is the sum of the drawdown contribution from each well.

$$(H - h) = \sum_{i=1}^{n} (H - h)_i \qquad (7.16)$$

Established mathematical expressions for the drawdown from an individual well can be applied to Equation 7.16 to estimate the drawdown at a given

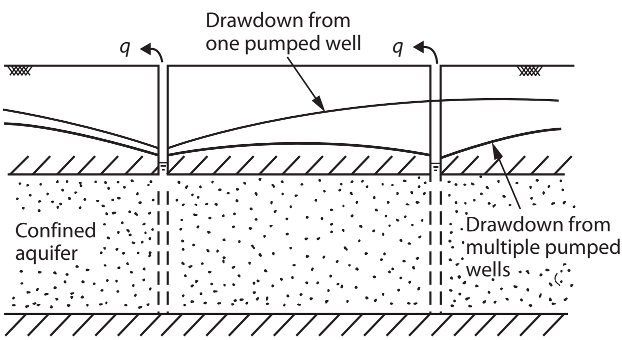


Figure 7.12 Superposition of drawdown from multiple wells.

point. For example, using the method of Theis (1935) in a homogeneous and isotropic confined aquifer of permeability k, thickness D, and storage coefficient S, the cumulative drawdown from n fully penetrating wells, each pumped at a constant rate q_i, at time t after pumping commenced is

$$(H-h)=\sum_{i=1}^{n}\frac{q_i}{4\pi kD}W(u_i) \tag{7.17}$$

where $W(u)$ is the Theis well function, values of which are tabulated in the work of Kruseman and De Ridder (1990), $u = (r^2S)/(4kDt)$, and r is the distance from each well to the point under consideration. For values of u less than approximately 0.05, the simplification by Cooper and Jacob (1946) can be applied, giving

$$(H-h)=\sum_{i=1}^{n}\frac{q_i}{4\pi kD}\left\{-0.5772-\ln\left[\frac{r_i^2S}{4kDt}\right]\right\} \tag{7.18}$$

In many aquifers, the condition of $u < 0.05$ is satisfied after only a few hours of pumping, which means that Equation 7.18 can generally be used for the analysis of groundwater control systems in confined aquifers.

Knowing the target drawdown ($H - h$) in the excavation area, these equations can be solved to determine the number, location, and yield of wells necessary to achieve the required drawdown. This also allows the total discharge flow rate (the sum of flow from all the wells) to be determined. This method is most suitable for systems of relatively widely spaced wells. It is mainly used for deep wells and, occasionally, for ejector systems; it is rarely used for wellpoint systems.

The following points should be considered when applying the method:

1. The method has been reliably applied to the estimation of drawdown within the area of excavation, away from the pumped wells themselves. Estimating the cumulative drawdown inside each well is more difficult because well losses may not be accurately known. If large well losses occur, the method is less reliable because the drawdown contribution becomes uncertain.
2. Application of the method requires that the aquifer parameters and well yields be estimated. In practice, the most reliable way to obtain suitable estimates is from the analysis of a pumping test. If pumping test data are not available, the estimated cumulative drawdowns should be treated with caution unless there is a high degree of confidence in the parameter values used in calculations. If a pumping test has been carried out, the graphical cumulative drawdown method

(described in the subsequent section) may be a more appropriate method of analysis.

3. It may be possible to obtain the required drawdowns in the proposed excavation using a few wells pumped at high flow rates, or a larger number of wells of lower yield. Similarly, varying the well locations around (or within) the excavation may produce significantly different drawdowns in the area of interest. In years gone by, investigating the effect of the various options was a tedious process. However, since the advent of widely available personal computers, it is possible to write routines or macros for spreadsheet programs to evaluate Equation 7.18, allowing many options to be rapidly considered. When evaluating the various options, it is vital that realistic well yields are used (see Section 7.8); otherwise, too many or too few wells will be specified.
4. Equations 7.17 and 7.18 include a term for the time since pumping began, so each cumulative drawdown calculation is for a discrete time t. The time used in calculation will depend on the construction program. If the program shows that a 2-week period is available for drawdown (between installation of the dewatering system and commencement of excavation below original groundwater level), then that case should be analyzed. However, in reality, there may be problems with the installation of a few wells and pumps, and thus, not all n wells will be pumping for the full 2-week period. It may be prudent to design on the basis of obtaining the target drawdown in a rather shorter time.
5. The assumptions inherent in Equations 7.17 and 7.18 (isotropic confined aquifer, fully penetrating wells, constant flow rate from each well) obviously will not apply in all cases. Provided that the basic aquifer conditions are confined or leaky, it may be possible to use Equation 7.17 for other conditions by substituting an alternative expression in place of the Theis well function $W(u)$. Kruseman and De Ridder (1990) provide well functions for a number of cases, including leaky aquifers, anisotropic permeability, partially penetrating wells, and variable pumping rates.

The cumulative drawdown method assumes that individual wells do not interfere significantly with each other's yield. For wells at wide spacings (greater than around 20 m) in confined aquifers (in which the aquifer thickness does not change with drawdown), interference is usually low. In such cases, the observed drawdowns are likely to be close to those predicted directly from the cumulative drawdown method. However, in general, observed drawdowns will be slightly less than predicted. It is not unusual for observed drawdowns to be between 80% and 95% of the calculated values, when applied using reliable parameters derived from pumping tests. To allow for this, the total well yield (or the number of wells to be installed) should be increased (by dividing by an empirical superposition factor J of 0.8–0.95). For example, the total system flow rate Q is determined from the sum of the individual flow rates q_i from n wells.

$$Q = \frac{1}{J}\sum_{i=1}^{n} q_i \tag{7.19}$$

The cumulative drawdown method is invalid in unconfined aquifers (or confined aquifers in which the drawdown is so large that local unconfined conditions develop). This is because the saturated thickness decreases as drawdown increases, making each additional well less effective compared with the initial wells. Although the method is theoretically invalid in unconfined conditions, where drawdowns are small (less than 20% of the initial saturated aquifer thickness), the method has been successfully applied using an empirical superposition factor J of 0.8–0.95. For greater drawdowns in unconfined aquifers, the cumulative drawdown method has been applied using empirical superposition factors of 0.6–0.8.

7.7.3 Cumulative drawdown analysis—Graphical method

If distance–drawdown data are available describing the aquifer response to the pumping of a single well, a graphical cumulative drawdown method can be used. This approach is based on the Cooper–Jacob straight line method of pumping test analysis (see Section 6.7) which uses Equation 7.18 expressed as

$$(H - h) = \sum_{i=1}^{n} \frac{q_i}{2\pi kD} \ln\left({R_0}/{r_i} \right) \tag{7.20}$$

where all terms are as described previously, apart from R_0, which is the radius of influence at time t. The equation is evaluated graphically and is used to obtain the total drawdown ($H - h$) at the selected location, resulting from a given array of wells, without the need to evaluate the aquifer parameters.

The method is described in detail by Preene and Roberts (1994) and involves the following steps:

1. Determine the target drawdown level in critical points of the excavation. Typically, critical points where drawdown is checked include the center and corners of the excavation. Normally, the target drawdown is a short distance (0.5–1 m) below the excavation formation level.
2. From the pumping test data, construct a drawdown–distance plot on semilogarithmic axes. Drawdowns recorded in observation wells at a given time after pumping commenced are plotted, and a best straight

line is drawn through the data (Figure 7.13a). For short-duration pumping tests, the data used are normally from the end of the test. The drawdown in the pumped well is normally ignored as it may be affected by well losses.

3. Convert each drawdown data point to a specific drawdown by dividing the drawdown by the discharge flow rate recorded during the test. A straight line is then drawn through the observation well data to obtain the design-specific drawdown plot (Figure 7.13b); this plot

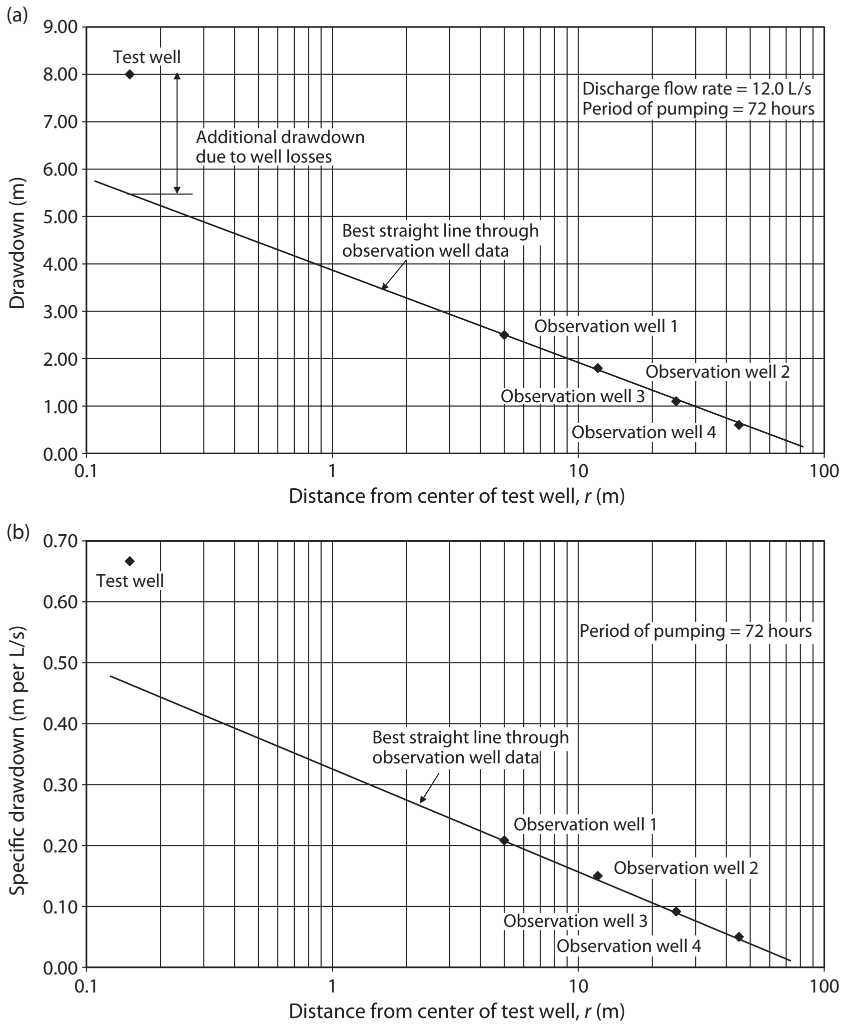


Figure 7.13 Cumulative drawdown analysis: graphical method. (a) Distance–drawdown plot. (b) Specific drawdown plot.

shows the drawdown which results from a well pumped at a unit flow rate.

4. Draw a plan of the excavation and groundwater lowering system, marking locations and the points on the well where drawdown is to be checked. Measure and record the distances from each well to each drawdown checking point.
5. Estimate the yield of each well in the system. This may be based on the pumping test results (based on step–drawdown data) or may involve the guidelines in Section 7.8.
6. At each drawdown checking location, calculate the drawdown which will result from the assumed set of well locations and yields. This is done using the specific drawdown plot (Figure 7.13b). The drawdown contribution $(H - h)_i$ from each well is calculated by reading the specific drawdown at the appropriate distance and then multiplying by the assumed well yield. The total calculated drawdown $(H - h)$ is the sum of the contribution from each well, multiplied by an empirical superposition factor J.

$$\left(H-h\right)=J\sum_{i=1}^{n}\left(H-h\right)_{i} \tag{7.21}$$

J is normally taken to be between 0.8 and 0.95 in confined aquifers. As with the theoretical cumulative drawdown method, the graphical method has been applied in unconfined aquifers where drawdowns are less than 20% of the initial saturated aquifer thickness. An example calculation in a confined aquifer (Preene and Roberts 1994) is shown in Figure 7.14; in that case, J was back-calculated to be 0.92.
7. The calculated drawdown at each checking location is compared with the target drawdowns from step 1. If the drawdown is insufficient, the calculation is repeated after having either changed well locations, increased the number of wells, or increased individual well yields. It is vital that the well yields assumed are achievable in the field. If the assumed yields are too large, the system will not achieve its target drawdowns.

Although the graphical method is most commonly used where site investigation pumping test data are available, the technique can also be used with the observational method. In this approach, one of the first wells in the groundwater lowering system is pumped on its own in a crude form of pumping test. Drawdowns are observed in the other dewatering wells (which are unpumped at that time); these data allow distance–drawdown plots to be produced. The cumulative drawdown calculations are then used to help with the decision-making progress to decide whether or not to install additional wells. As each new well is installed and pumped, further drawdown data are collected and the predicted drawdowns compared with

Estimation of drawdown at Well 8 due to pumping on four other wells

Well No.	*Discharge flow rate per well (L/s)*	*Distance to Well 8 (m)*	*Specific drawdown (m per L/s)*	*Calculated drawdown (m)*
1	8.5	82	0.079	0.67
2	8.5	100	0.072	0.60
6	11.0	50	0.082	0.91
7	11.0	20	0.103	1.13
Total flow rate	39.0		Total drawdown at Well 8	3.31

Actual drawdown recorded at Well 8 after 44 hours = 3.06 m

Therefore, drawdown achieved is 3.06/3.31 = 92 per cent of calculated cumulative drawdown

Figure 7.14 Case history of cumulative drawdown calculation. (Data from Preene, M. and Roberts, T.O.L., *Groundwater Problems in Urban Areas* (edited by W.B. Wilkinson). Thomas Telford, London, 121–133, 1994: Reproduced by permission of ICE Publishing.)

the actual values. In these cases, it is often found that, as each additional well becomes operational, the empirical superposition factor *J* reduces further, as interference between wells increases.

7.7.4 Estimation of flow rates where cutoff walls are present

The analytical methods presented in Sections 7.7.1 through 7.7.3 are applicable to groundwater flowing directly to excavations where water can enter both the sides and the base of the excavation. However, in some situations, a low-permeability cutoff wall may be used, either as part of the excavation support system or to reduce groundwater inflows to the excavation (see Section 5.7). An example of this geometry is shown in Figure 5.2. For these cases, the widely used "equivalent well" model of groundwater flow to an excavation is invalid. It is necessary to use analytical methods which take account of vertical flow. Alternatively, these geometries can be analyzed using numerical groundwater models (see Section 7.10).

Kavvadas et al. (1992) developed an analytical solution for groundwater flow into excavations, where cutoff walls are present at the sides of an excavation, for the geometry shown in Figure 7.15a. The solution is based on the following assumptions:

- The width of the excavation is *b*, with identical cutoff walls on both sides;
- The excavation is of length *a* and is long in comparison to its width (which means that radial effects at the end of the excavation can be neglected);

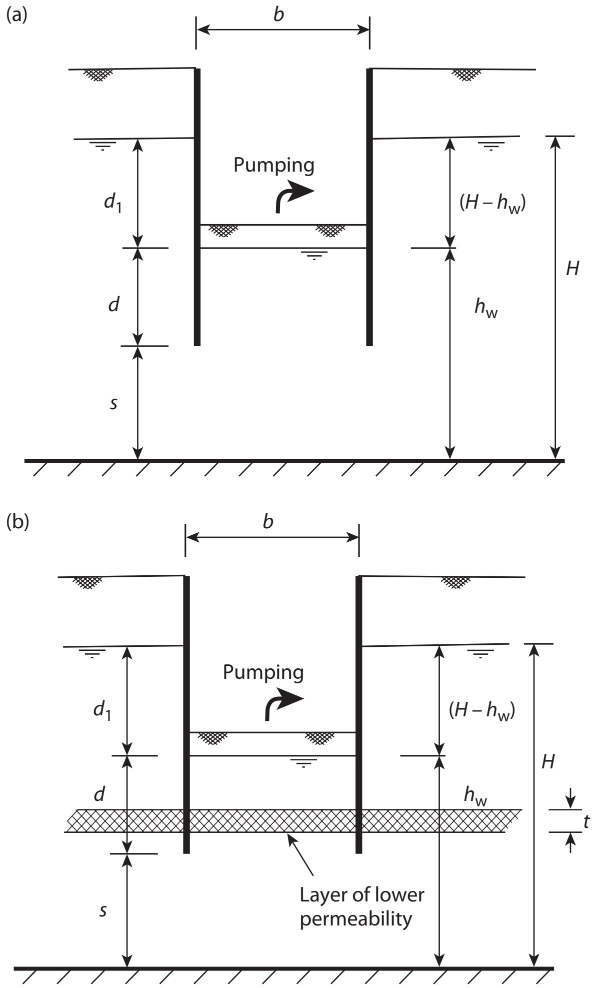


Figure 7.15 Groundwater inflow to excavation where cutoff walls are present. Uniform soil conditions (a) where a lower permeability layer is present (b).

- The soil or rock is assumed to be homogeneous and be of isotropic permeability k;
- The cutoff wall is impermeable and penetrates to depth d below the lowered groundwater level in the excavation (assuming groundwater level is lowered below formation level);
- The aquifer has an impermeable stratum at depth. The base of the cutoff wall is a distance s above the impermeable base of the aquifer; and
- Pumping from the excavation does not lower the external groundwater level.

The solutions for flow rate q per unit length of excavation and maximum upward hydraulic exit gradient i_{max} in the base of the excavation are

$$q = 0.85k(H - h_w)[1 - (0.2)^{s/0.5b}]\left(\frac{d}{0.5b}\right)^{-0.5}\left(\frac{d_1}{0.5b}\right)^{-0.125} \tag{7.22}$$

$$i_{max} = 0.5\left(\frac{H - h_w}{d}\right)[1 - (0.2)^{s/0.5b}]\left(\frac{d}{0.5b}\right)^{-1/3}\left(\frac{d_1}{0.5b}\right)^{-0.125} \tag{7.23}$$

where H, h_w, and d_1 are as shown in Figure 7.15a:

H = initial water table level in aquifer
h_w = lowered water table level in aquifer
$(H - h_w) = d_1$ = drawdown in excavation.

For a long excavation (neglecting flow to the ends of the excavation) the total flow rate can be estimated by multiplying the flow rate per unit length q from Equation 7.22 by the excavation length a.

The analytical methods presented in Equations 7.22 and 7.23 are applicable to cases in which cutoff walls are used in unconfined aquifers to reduce inflows to the excavation. Because the method assumes that pumping does not cause any external drawdowns, the method is probably conservative and is likely to provide estimates of flow rate more applicable to the early phases of pumping, before storage release (see Section 7.7.6) has occurred, rather than long-term pumping rates.

Kavvadas et al. (1992) also provide a solution for the case in which a continuous horizontal layer (of thickness t) of lower permeability material is present beneath the excavation formation level and above the toe of the cutoff wall (Figure 7.15b). In that case, the solutions for flow rate q_r per unit length of excavation and maximum upward hydraulic exit gradient i_{maxr} in the base of the excavation are

$$q_r = q/\mu \tag{7.24}$$

$$i_{maxr} = i_{max}/\mu \tag{7.25}$$

$$\mu = 1 + 0.25\left(\frac{t}{d}\right)\left(\frac{k}{k_r} - 1\right) \tag{7.26}$$

where

q_r = flow rate per unit length of excavation for case when low permeability layer is present;
i_{maxr} = maximum hydraulic exit gradient for case when low-permeability layer is present;

q = flow rate per unit length of excavation for uniform permeability case (Equation 7.22);
i_{max} = maximum hydraulic exit gradient for uniform permeability case (Equation 7.23);
t = thickness of the low-permeability layer;
k_r = permeability of the low-permeability layer; and
k = permeability of the remainder of the aquifer.

7.7.5 Estimation of flow rates into tunnels

The analytical methods presented in Sections 7.7.1 through 7.7.4 are applicable to the typical geometries of dewatering systems around trenches or excavations. They are not directly applicable to problems in which groundwater inflow to tunnels must be estimated. The following section discusses some aspects of groundwater inflow to tunnels.

The potential for groundwater inflows exists whenever tunnels are driven below groundwater level through permeable soil or rock strata. The tunnel acts as a drain, with water flowing into the tunnel by gravity. The quantity of water inflow may affect the methods and equipment used during construction. Relatively small inflows may simply be a cause of discomfort and inefficiency in excavation and tunnel lining operations. More substantial inflows may have a huge influence on the works, in extreme cases, giving an actual risk of inundation of the tunnel.

Inflows can be reduced or managed by various methods, including the use of compressed air, tunnel-boring machines, grouting, and other forms of ground treatment, or by pumping or other methods (see Section 5.6). During the planning of tunneling projects, it is necessary to estimate potential water inflows as part of the decision-making process. Only then can appropriate ground treatment and tunneling methods be specified. However, groundwater flow conditions around a tunnel are complex. Accurate prediction of inflows is difficult for a number of reasons, including

- The natural inhomogeneity of the permeability of soils (due to layering and change in material type) or of rocks (due to variations in fracture spacing, opening, and orientation).
- The complex time-dependent interaction between the tunnel and the surrounding ground. For example, even the rate of tunnel advance will affect the inflows because a slowly advancing tunnel may drain the ground ahead of it, reducing inflows.
- The effect of tunnel construction methods. If tunnels are lined immediately after excavation, inflows may be restricted to areas immediately around the working face. If the tunnel is to remain unlined, or is lined in one operation late in the project, inflows are possible over a much wider area.

These factors mean that, even when comprehensive site investigation data are available, experience of the local geology and construction experience will be useful when estimating inflows. This section presents an analytical approach which can be used in support of engineering judgment.

Use of analytical methods requires some estimation of the permeability of the soil or rock through which the tunnel is to be driven. In all cases, the description of the soil type or rock type, rock quality, and rock fracture descriptors are vital to allow engineering judgment to be applied and to highlight any "rogue" readings within the permeability data (see Section 6.7).

Groundwater inflow to tunnels can take two principal forms:

- Flow from porous medium such as a granular soil (e.g., sand or gravel) or from a porous rock in which the flow through discrete fractures does not dominate over flow through the rock mass as a whole. The medium may be more or less isotropic and homogenous. In most such cases, the flow of groundwater will be laminar, and Darcy's law will apply. The permeability k of the medium, and groundwater recharge conditions are important controlling parameters of inflows.
- Flow from discrete fissures, fractures, discontinuities, or solution features, as might be found in competent rock. Such flow may be turbulent and Darcy's law would not apply. Inflows are controlled by the discontinuity aperture size and spacing, the extent of the fissure system and any recharge conditions. Any inflow calculations require the definition of likely discontinuity conditions.

Laminar flow of groundwater through porous media is described by Darcy's law (Equation 3.2). The formulae in this section are derived from Darcy's law. Darcy's law can apply to groundwater flow through fractured rocks, provided that the fracture-induced permeability is assumed to give the same effect as a homogenous porous medium. This might be the case if the fractures are uniformly closely spaced. In both soils and rocks, if very high inflows occur, turbulent flow may develop just outside the edge of the tunnel excavation, and Darcy's law will be invalid in that area.

Whatever method of analysis is used, some form of sensitivity or parametric analysis may be appropriate to assess the likely range of inflows, based on the available data. These estimates may help determine whether ground treatment to reduce permeability (such as grouting) will be essential, or should be included only as a contingency. It is useful to consider the sensitivity of calculated inflows to selected parameters.

Tunnel diameter: A number of studies, including the work of Fitzpatrick et al. (1981), have highlighted that inflow rates are not especially sensitive to tunnel diameter, and inflows are not dissimilar for small and large tunnels. Wrench and Stacey (1993) state that this is supported by the observation that inflow to pilot bores is often as large as into the final tunnel.

Depth below groundwater level: This has an important effect on calculated inflows, which increase rapidly as the depth is increased.

Soil or rock permeability: The permeability of the soil or rock is critical to the calculation of inflows. This inflow comes not only from the level of the tunnel itself, but also from the soil or rock above and below the tunnel—this should influence the planning of site investigation permeability testing. In many fissured rocks, the overall permeability reduces with depth, and this will affect tunnel inflows. Wrench and Stacey (1993) suggest that simple calculations may overestimate inflows if the permeability values used are based solely on the level of the tunnel.

Groundwater recharge and storage: If the tunnel is driven rapidly through a stratum in which there is sufficient groundwater recharge and storage (either from precipitation or flow from other strata) to prevent groundwater levels from being lowered, the tunnel will not significantly disturb drawdown water levels. If the water table is not lowered, inflow to the tunnel will remain steady and will not decline with time (Figure 7.16a). Conversely, if there is limited groundwater recharge and storage, especially if the tunnel is driven slowly, allowing more time for water to drain, groundwater levels will be lowered (Figure 7.16b). Inflows will decline from their initial values as a “distance of influence” develops away from the tunnel. The long-term water inflows are unlikely to be zero but will stabilize at some flow rate in balance with the available storage and recharge.

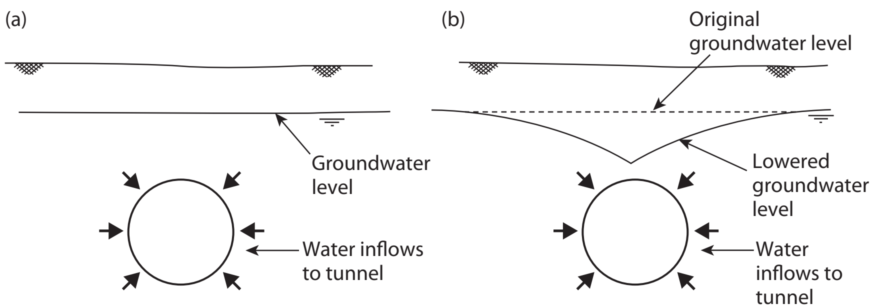


Figure 7.16 Effect of tunnels on groundwater levels. (a) No drawdown of groundwater levels. The groundwater storage or recharge is sufficiently large to prevent any significant lowering of groundwater levels as a result of water draining into the tunnel. (b) With the drawdown of groundwater levels, water draining into the tunnel results in lowering of groundwater levels.

Steady-state groundwater inflows Q to tunnels in conditions in which Darcy's law is valid (i.e., where the flow is not concentrated in a small number of discrete features) can be estimated using the equation by Goodman et al. (1965), which is based on the geometry shown in Figure 7.17 and the assumptions stated below:

$$Q = \frac{2\pi k H x}{\ln(4H/D)} \tag{7.27}$$

where k = the permeability of the soil or rock, D is the diameter of the tunnel, x is the length of the tunnel being assessed, and H is the head difference between the groundwater level and the tunnel axis. Goodman's equation is based on the following assumptions:

- A tunnel of infinite length
- Steady-state conditions
- Soil or rock of homogenous and isotropic permeability
- No drawdown of the groundwater level
- Flow occurs to the cylindrical face of the tunnel only; no allowance is made for the flow to the end of tunnel drive (i.e., the working face)

Because of the limitations in the simplifying assumptions, equations such as that of Goodman should be used with caution, but can provide order of magnitude estimates of likely tunnel inflows. Methods for estimating groundwater inflows to rock tunnels are discussed in more detail in the work of Heuer (1995, 2005). Because of the complex geometry of many tunnel construction projects, numerical groundwater modeling (see Section 7.10) can be a suitable tool to develop groundwater inflow estimates for different stages of tunnel construction.

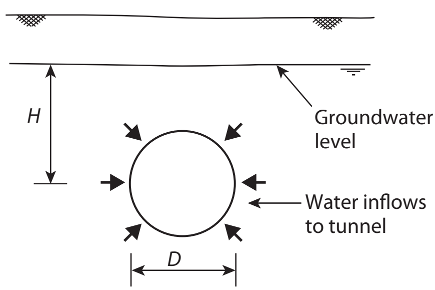


Figure 7.17 Groundwater inflow to tunnels.

7.7.6 Storage release and uprating of pumping capacity

Many of the methods used to calculate discharge flow rates assume that "dewatered" conditions have developed and that a zone of influence of drawdown exists in the aquifer around the groundwater lowering system. To reach this condition, water must be released from storage in the aquifer within the zone of influence (see Section 3.5). This means that, during the initial period of pumping, before the steady state is approached, an additional volume of water must be pumped.

In confined aquifers, the quantity of water from storage release is small and is only significant during the first few hours of pumping. Its effect on the necessary pumping capacity is often neglected. In contrast, in unconfined aquifers, the water from storage may be significant and may persist for several weeks or more, dependent on the aquifer permeability and the pumping rate. Powers et al. (2007) suggest that storage release is likely to be a major issue in permeable unconfined aquifers where the proposed discharge rate is more than 60–70 L/s.

Storage release means that either a system designed with a capacity equal to the steady-state flow rate will take longer than anticipated to achieve the target drawdown or the design system flow rate should be increased above the steady-state estimate to deal with water from storage and to ensure that drawdown is achieved within a reasonable period.

The release of water from storage has the same effect as reducing the distance of influence used in calculation. If the "long-term" distance of influence is used in design, drawdown may only be achieved slowly. Alternatively, if the designer uses a "short-term" distance of influence in design, drawdown may be achieved rapidly, but the system may be overdesigned in the long term. This is only likely to be an issue in high-permeability unconfined aquifers.

If the distance of influence used in design is taken from analysis of short-term pumping test data (see Section 6.7), this will include storage release, and calculated flow rates are likely to achieve drawdowns fairly rapidly. This also applies to systems designed by the graphical cumulative drawdown method, where pumping test data are used directly.

If no pumping test data are available and the distance of influence is estimated from empirical formulae such as Equations 7.13 through 7.15, then judgment must be used where distances of influence of several hundred meters are predicted in high-permeability soils. Systems with pumping capacities designed on that basis will be able to cope with steady-state inflows, once the zone of influence has developed. However, they may be overwhelmed by storage release during the early stages of pumping, and drawdown may take a long time to be achieved, leading to delays in the construction program. Designers sometimes overcome this problem by using rather smaller distances of influence which predict higher flow rates; this helps ensure that drawdown is achieved in reasonable time.

If the distance of influence used in the design has been estimated from Equations 7.13 through 7.15, the following equations can be used to crudely estimate the time t, which would be required for this distance of influence to develop. The equations are for radial flow (according to Cooper and Jacob 1946)

$$R_0 = \sqrt{\frac{2.25kDt}{S}} \tag{7.28}$$

and for plane flow (according to Powrie and Preene 1994a)

$$L_0 = \sqrt{\frac{12kDt}{S}} \tag{7.29}$$

where D is the aquifer thickness, k is the permeability, and S is the storage coefficient. Strictly, Equations 7.28 and 7.29 are only valid in confined aquifers, but can be used in unconfined aquifers where the drawdown is not a large proportion of the original saturated thickness. Typical values of specific yield (approximately equal to S in an unconfined aquifer) are given in Table 3.2.

If the design distance of influence will take a long time (more than a few days) to develop, it is possible that the system should be designed assuming a smaller R_0 or L_0 (values typically used in high-permeability soils are in the range of 200–500 m). Changing the design in this way would increase the required pumping capacity of the system. This would allow the water released from storage to be handled by the system and would result in the drawdown within the excavation area being achieved within a reasonable length of time.

Storage depletion is perhaps of most significance in high-capacity dewatering systems operating for long periods of time, as might occur with a dewatering system for an open-pit mine, or for a permanent dewatering system (Chapter 14). On such schemes, if hydrogeological analysis indicates that the pumped flow rates will initially be high, but will be reduced after several weeks or months of pumping as aquifer storage depletion occurs, one option is to use large pumps for initial pumping and then swap them for smaller pumps. The labor cost of the pump changeout operation will be recovered in reduced energy usage over future years. This approach is described in more detail by Rea and Monaghan (2009).

7.7.7 Other methods

Occasionally, other methods are used to estimate steady-state flow rates. Flow net analyses are sometimes used to model flow patterns not amenable to simplification as equivalent wells or slots. A flow net is a graphical representation of a given two-dimensional groundwater flow problem and its associated boundary conditions. Flow nets are one of the common forms of output of numerical groundwater models (see Section 7.10). However, as described by Cedergren (1989), hand-sketching of flow nets can be used to obtain solutions to certain flow problems, considered either in plan or cross-section, for isotropic or anisotropic conditions. Typical problems where flow nets are used include seepage into excavation or cofferdams in which the presence of partial cutoff walls alters the groundwater flow paths; see the work of Williams and Waite (1993).

Very rarely, physical models or electrical resistance or resistance-capacitance analogues (see the work of Rushton and Redshaw 1979) are used to analyze groundwater flows. In the past, they were used more commonly to analyze complex problems, but in recent times, advances in numerical modeling methods have made these techniques largely obsolete. A rare recent application of the use of electrical analogues is described by Knight et al. (1996).

7.8 SPECIFICATION OF WELL YIELD AND SPACING

Having determined the total required pumping rate, the next step is to determine the yield and spacing of wells (be they wellpoints, deep wells, or ejectors).

7.8.1 Well yield

Each well must be able to yield sufficient water so that all the wells in concert can achieve the required flow rate and, hence, the required drawdown.

Water enters a well where the well screen penetrates below the lowered water table in an unconfined aquifer, or where it penetrates a saturated confined aquifer. This depth of penetration is known as the "wetted screen length" and is an important factor in the selection of well depths to achieve adequate yield (Figure 7.18).

In theoretical terms, the yield q into a well can be described (from Darcy's law) by

$$q = 2\pi r l_w k i \tag{7.30}$$

where

r = the radius of the well borehole (not the diameter of the well screen). This assumes the well filter media are of significantly greater permeability than the aquifer

l_w = wetted screen length below the lowered groundwater level

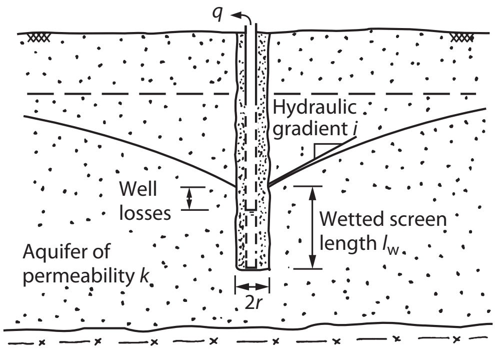


Figure 7.18 Wetted screen length of wells

k = aquifer permeability
i = hydraulic gradient at entry to the well

The designer has no control over aquifer permeability, but can vary the length and diameter of the well within limits determined by the geology of the site and the availability and cost of well drilling equipment. Experience suggests that any well will have a maximum well yield, beyond which the well will "run dry," that is, the water level in the well will reach the pump inlet or suction level, preventing further increases in flow rate.

In 1928, Sichardt published an article entitled *Drainage Capacity of Wellpoints and its Relation to the Lowering of the Groundwater Level*, which examined the yield of pumped wells based on the records of numerous groundwater lowering projects. He determined empirically that the maximum well yield is limited by a maximum hydraulic gradient, i_{max}, which can be generated in the aquifer at the face of a well. The Sichardt limiting gradient is generally taken as relating the aquifer permeability k (in meters per second) to the maximum hydraulic gradient at the face of the well by

$$i_{max} = \frac{1}{15\sqrt{k}} \tag{7.31}$$

This is the theoretical maximum amount of water that a *well* can yield—in very high-permeability aquifers, the potential well yield may be so large that the actual flow rate is controlled by the *pump* rather than the well.

The Sichardt gradient is probably reasonable to estimate the maximum well yield in a relatively high-permeability aquifer (k greater than approximately 1×10^{-4} m/s). However, the work of Preene and Powrie (1993), who analyzed well yields in a large number of dewatering systems in fine-grained soils, has suggested that Equation 7.31 may not be appropriate for lower permeability aquifers. The Sichardt gradient may overestimate hydraulic gradients (and hence well yields) in soils of permeability less than approximately 1×10^{-4} m/s. The work of Preene and Powrie indicated that in lower permeability soils, hydraulic gradients were generally lower than 10 and that an average of six was not unreasonable. These two approaches are combined in Figure 7.19 to provide the maximum well yield per unit wetted screen length for wells of various diameters of bored hole. Yield per unit length for other diameters can be calculated using Equation 7.31 and a limiting hydraulic gradient appropriate to the aquifer permeability.

Figure 7.19 can be used to design deep well or ejector systems. Wellpoint systems (or systems of ejectors with short screens at very close spacings) are analyzed rather differently, as will be described later.

For wells at wide spacings in an aquifer that extends to some depth below the excavation, once a well diameter is assumed, Figure 7.19 can be used to determine the minimum wetted screen length (below the lowered water level) needed to obtain the total flow rate from the deep well system. This would then allow the number of wells and wetted depth per well to be estimated. For example, if 120 m of wetted screen length was estimated, this could be equivalent to twelve wells with 10 m of wetted screen each, or eight wells with 15 m of wetted screen each, and so forth. Obviously, a check then needs to be made that the originally assumed well diameter is large enough to accommodate well screens and pumping capacity to

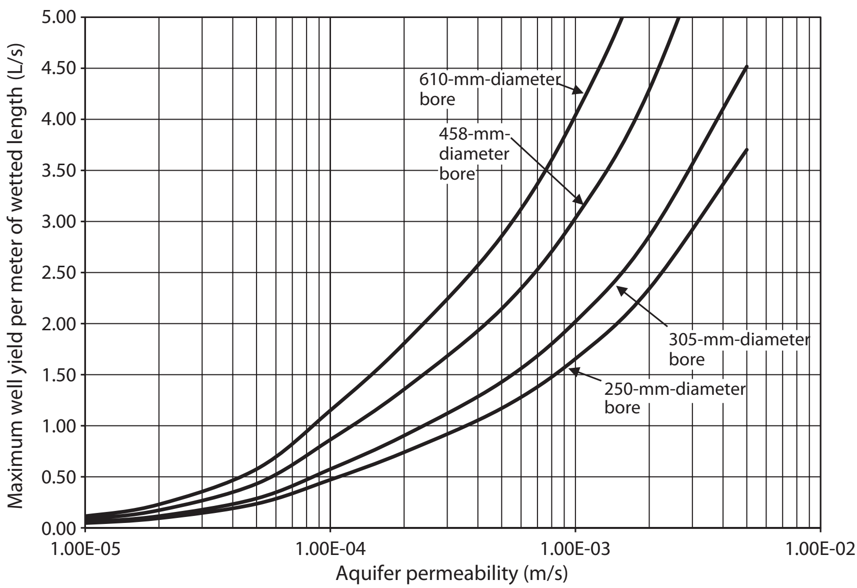


Figure 7.19 Maximum yield per unit wetted length of wells.

produce the design yield. If necessary, the well diameter must be increased. Guidance on the size of deep wells and ejector wells required to produce given discharge rates per well are given in Chapter 10 and Section 11.2, respectively.

Estimation of well yield is a point in design where judgment and experience can be vital, so the designer should consider the following:

1. The wetted screen length, l_w, will be rather less than the penetration of the well below the "general" lowered water level. This is because of the additional drawdown around each well due to each individual cone of depression (Figure 7.18). The difference between the wetted depth and the excavation drawdown level can be estimated from pumping test results. If no pumping test data are available, l_w must be estimated based on engineering judgment.
2. The yields calculated by the methods described above are theoretical maximums and might not be achieved in practice. Well yields may be reduced by the use of inappropriate filter material or poor development. The method of drilling will also influence yield, with jetted boreholes generally being more efficient than rotary or cable tool percussion drilled holes. Experience also suggests that two competent drillers using the same methods and equipment on holes a few meters apart can produce wells with wildly differing yields. There is no conclusive explanation for this phenomenon, but it is likely to be related to the precise way each driller uses the bits, casing, and drilling fluids and how that affects a thin layer of aquifer just outside the well.
3. There are certain rules of thumb about well depth, built-up over many years of practice, and the results of calculations should be compared with these to check for gross errors. If the aquifer extends for some depth below the base of the excavation being dewatered, the wells should be between 1.5 and 2 times the depth of the excavation. Wells significantly shallower than this are unlikely to be effective unless they are at very close spacing (analogous to a wellpoint system).
4. If the geology does not consist of one aquifer that extends to great depth, this may affect well depth and will restrict the flexibility of the designer in specifying l_w. If the aquifer is relatively thin, screen lengths will be limited by the aquifer thickness and a greater number of wells will be required. If there is a deeper permeable stratum that, if pumped, could act to underdrain the soils above, it may be worth deepening the wells (beyond the minimum required) to intercept the deep layer (see Section 7.6.2).
5. It is always prudent to allow for a few extra wells in the system over and above the theoretically calculated number. Typically, for small systems, at least one extra well is provided, or for larger systems, the number of wells may be increased by around 20%. This allows for

some margin for error in design or ground conditions but also means that the system will be able to achieve the desired drawdown if one or two wells are nonoperational because of maintenance or pump failure.

In a similar manner, the potential yield of a wellpoint or an ejector installed in a jetted hole can be assessed using Equation 7.31, and a limiting hydraulic gradient appropriate to the aquifer permeability. Figure 7.20 shows the theoretical maximum yield of 0.7-m-long wellpoint screens. Yield from longer or shorter screens can be estimated pro rata. The figure shows that even in very high-permeability soils, a conventional wellpoint is unlikely to yield more than around 1 L/s (special installations of larger diameter and longer screen length may yield more, but such applications are rare). A maximum possible wellpoint yield of 1 L/s is a useful practical figure for the designer to remember.

Figure 7.20 is based on a disposable wellpoint installed in a 200-mm-diameter sand filter, and a self-jetting wellpoint installed in a jetted hole of 100–150 mm diameter. When dealing with jetted wells, the diameter of the jetted hole may be uncertain, requiring further judgment to be used. In theory, a wellpoint system could be installed in an analogous manner to a deep well system, by determining the necessary total wetted screen length and the corresponding number of wellpoints and then adding additional wells as a contingency. In practice, this process is rarely carried out. Wellpoint

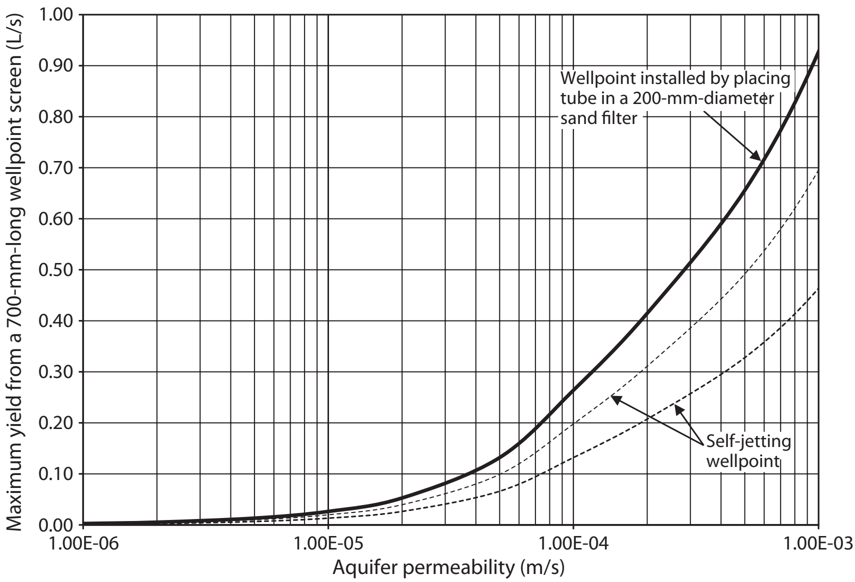


Figure 7.20 Maximum yield of wellpoints.

equipment is almost always used in one of a limited number of standard spacings. Methods of selecting wellpoint spacings are described below.

The estimated well yield can be used to select the capacity of the pumping equipment. For deep wells and ejectors, the pumping equipment is located in each well and is sized directly from the well yield. For wellpoint systems, one pump acts on many wellpoints in concert. The designer has the choice of a few larger pumps, or a greater number of smaller units (see Section 9.7). When estimating the required pump capacity from calculated steady-state flow rate, an additional allowance must be made for the greater flow rate from storage release during the initial period of pumping (see Section 7.7.6).

7.8.2 Number of wells and well spacing

As described previously, for deep well and ejector systems, the number of wells required can be determined from the total discharge flow rate, divided by the predicted well yield, with some additional wells added as a contingency. This will then allow the average well spacing (the distance between adjacent wells) to be determined. Because well spacings of most groundwater lowering systems fall within relatively narrow ranges, comparison of the "design" spacing with typical values can be a useful way of verifying a design.

For all systems, the well spacing chosen will influence the time required to lower groundwater levels to the target drawdown. In general, the closer the well spacing, the quicker drawdown will be achieved. Because time is often as important a factor as cost in construction, groundwater lowering systems are often installed with wells at rather closer spacing than is theoretically necessary, to ensure drawdown is achieved within a few days or weeks.

Typical spacing of deep wells are in the range of 5–100 m, although the great majority of systems use spacings in the range of 10–60 m between wells.

In aquifers of moderate to high permeability (where maximum well yields are relatively high), the designer has the flexibility of choosing a larger number of closely spaced low-yield wells, or a smaller number of widely spaced high-yield wells. When potential well yields are large, high-capacity pumps may not be available, and the output from each well may be controlled by the pump performance; this should be considered when estimating well numbers and spacings. If the aquifer extends to great depth, it may be possible to use relatively few very high-capacity wells of very great depth at spacings of several hundred meters. This approach would require extensive pump test data, backed up by numerical modeling.

In low-permeability soils, the well yield will be relatively low, and the option of a small number of widely spaced high-yield wells is not available to the designer. To achieve the total discharge flow rate, the wells will be at close spacings. If a well spacing of less than around 5–10 m is suggested by the design, or if the well yield is less than around 0.7 L/s, an ejector system could be considered as an alternative to deep wells.

Ejector systems can be used in two ways.

1. They can be used in soils of moderate permeability as an alternative to low-yield deep wells, or as “deep wellpoints” to achieve drawdowns of more than 5–6 m. When used in this way, well spacings are similar to those used for deep wells (5–10 m) or wellpoints (1.5–3 m).
2. In soils of low-permeability, they can be used as a vacuum-assisted pore water pressure control method. Because of the limited area influenced by each well, ejectors tend to be installed at close spacings (1.5–5 m) and operate at only a fraction of their pumping capacity.

In contrast to deep well and ejector systems, wellpoint spacings are rarely selected on a yield per well basis. The number of wellpoints is determined by first selecting the wellpoint spacing from a fairly narrow range; the number of wellpoints is then determined from the length of the line or ring of wellpoints. Typical wellpoint spacings are in the range of 0.5–3.0 m (see Table 9.1 for spacings categorized by soil type). Closer spacings tend to reduce the time to achieve drawdown compared with wider spacings. Wellpoint spacing is controlled by different factors in soils of high and low-permeability.

1. In high-permeability soils (such as coarse gravels), the total discharge flow rate will be large. Each wellpoint may be operating at high yields. A rule of thumb is that a conventional wellpoint cannot yield more than approximately 1 L/s, and the spacing is sometimes based on reducing the average wellpoint yield of less than 1 L/s. Wellpoints tend to be installed at close spacings in high-permeability soils. If a spacing in the range of 0.5–1.0 m is suggested by design calculations, wellpoint dewatering may not be the most appropriate method. High-capacity deep wells at close spacings might be considered as an alternative.
2. In low-permeability soils (such as silty sands), the total flow rate will be low. Consideration of the maximum wellpoint yield may suggest that fairly wide (5–10 m) spacings may be possible. In reality, because of the limited area influenced by each wellpoint, such a system is likely to perform poorly and a very long time may be required to achieve drawdown. The solution is to install wellpoints at closer spacings to 1.5–3.0 m.

7.9 OTHER CONSIDERATIONS

Although the prime concern of most groundwater lowering designs is to estimate the steady-state flow rate and the corresponding number and yield of wells that will be required, other issues sometimes need to be addressed during design.

7.9.1 Estimation of time-dependent drawdown distribution around the well

As pumping continues, the zone of influence around a dewatering system will increase with time. This means that the area affected by drawdown will expand, initially rapidly, and then progressively more slowly as time passes. This is clearly a complex problem, and a complete solution probably requires a suitably calibrated numerical model. However, there are some simpler methods that can be used to estimate the drawdown pattern around an excavation at a given time. These methods are outlined in this section.

In dewatering design, the time-dependent drawdown distribution may be required for the following reasons:

1. To determine how far the zone of significant settlements will extend during the period of pumping. This is useful if the effect of potential environmental impacts (see Chapter 15) is being assessed. This normally involves producing a plot of the drawdown versus distance at time t after pumping began. Successive plots at greater values of t can show the development of the zone of influence. Each plot of drawdown versus distance at a given time is known as an isochrone.
2. To determine the time required to achieve the target drawdown in a particular part of the excavation. This is normally only an issue in low-permeability soils where it may influence the construction program. In moderate to high-permeability soils, experience has shown that most appropriately designed systems should achieve the target drawdown within 1–10 days of pumping.

In many cases, these calculations are unnecessary because the time to achieve drawdown and the risk of side effects is not a major concern. In other cases, the methods presented here can be used to *approximately* determine the time-dependent drawdown distribution. All these analyses assume that the aquifer is homogenous and that no recharge or barrier boundaries are present within the zone of influence (i.e., all pumped water is derived purely from storage release). Numerical modeling should be considered where aquifer boundaries are likely to be present.

If the groundwater lowering system consists of relatively widely spaced wells, then cumulative drawdown methods can be used. The theoretical approach (Equation 7.16) can allow the predicted drawdown at a selected point to be plotted against time. Alternatively, the distance–drawdown profile (or isochrone) can be determined at a given time after pumping commences. By repeating the calculation for different times, a series of isochrones, showing the drawdown pattern at each time interval, can be produced.

If the system consists of closely spaced wells in a regular pattern, the system can be analyzed as an equivalent well or slot. Analyses which assume that constant drawdown in the well or slot can then be used. The constant drawdown assumption is a reasonable one for most dewatering systems (apart from during the first few hours of pumping) and has given acceptably accurate results in practice.

For horizontal plane flow to an equivalent slot in a confined aquifer, the drawdown curve can be expressed as a parabola (Powrie and Preene 1994a); shown in dimensionless form in Figure 7.21. The drawdown s at distance x from the slot can be determined from this figure provided the drawdown s_w at the slot and the distance of influence L_0 are known. The drawdown at the slot is normally taken to be the same as the drawdown inside the excavation (not the drawdown in individual wells), and L_0 is estimated from

$$L_0 = \sqrt{\frac{12kDt}{S}} \tag{7.32}$$

where D is the aquifer thickness, k is the permeability, S is the storage coefficient, and t is the time since pumping began.

For horizontal radial flow to an equivalent well in a confined aquifer, a time-dependent solution was developed by Rao (1973); this is plotted in

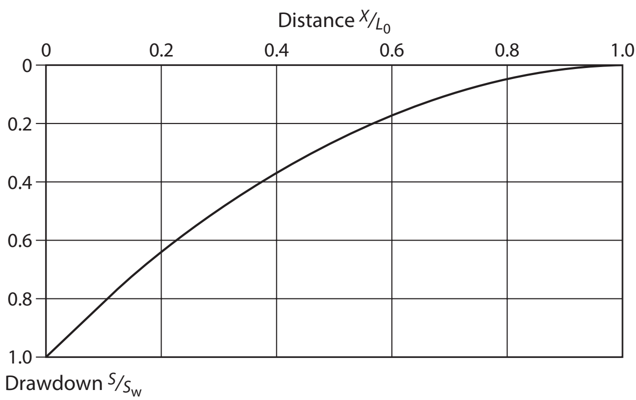


Figure 7.21 Dimensionless drawdown curve for a horizontal plane flow to an equivalent slot.

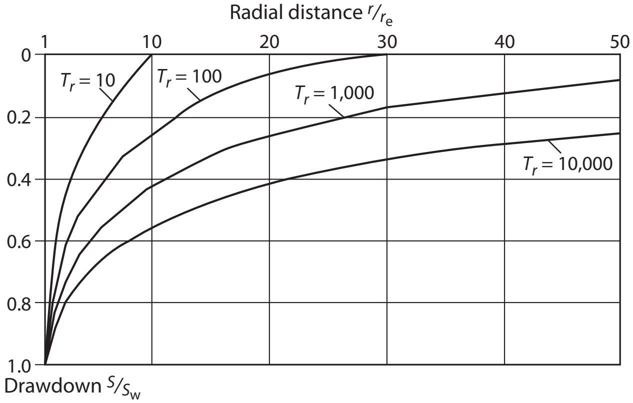


Figure 7.22 Dimensionless drawdown curve for a horizontal plane flow to an equivalent well. (After Powrie, W., and Preene, M., *Géotechnique*, 44, 1, 83–100, 1994.)

Figure 7.22. The drawdown s at radius r from the center of the well can be determined from this figure provided the drawdown s_w at the slot and the time factor T_r are known.

$$T_r = \frac{kDt}{Sr_e^2} \tag{7.33}$$

where r_e is the radius of the equivalent well, and all other terms are as defined previously.

These methods are only theoretically valid in confined aquifers, but can be used without significant error in unconfined aquifers where the drawdown is not a large proportion of the original saturated thickness.

Equations 7.32 and 7.33 use the storage coefficient S, which is appropriate for soils of moderate to high permeability. These methods can also be used where pore water pressure control systems are used in low-permeability soils, but the drainage characteristics of the soils may be expressed in terms of c_v, the coefficient of consolidation.

$$c_v = \frac{kE_0'}{\gamma_w} = \frac{kD}{S} \tag{7.34}$$

where E_0' is the stiffness of the soil in one-dimensional compression and γ_w is the unit weight of water. For applications in low-permeability soil, Equation 7.34 can be substituted into Equations 7.32 and 7.33.

7.9.2 Estimation of groundwater lowering-induced settlements

Ground settlements are an unavoidable consequence of the effective stress increases that result from groundwater lowering. In most cases, the settlements are so small that there is little risk of damage or distortion to nearby buildings. However, if compressible soils (such as peat or normally consolidated alluvial clays and silt) are present, it is possible that damaging settlements may occur.

If the conceptual model has identified the presence of significant thicknesses of compressible strata within the zone of influence, it will be necessary to consider the magnitude of ground settlements which may result. Methods for estimating the settlements that will result from a given drawdown are outlined in Section 15.4. The drawdown which may be expected beneath individual structures can be estimated by numerical modeling or from the drawdown distributions given earlier in this section.

It is important to remember the aim of settlement assessments—to assess the risk of damage to structures and services, rather than to try and predict settlements to the nearest millimeter. This latter aim would be difficult to achieve because the stiffness and consolidation parameters of compressible soils are rarely known with sufficient accuracy. Predicted degrees of damage, corresponding to various levels of settlement, are given in Section 15.4. That section also discusses the methods used to mitigate or avoid settlement damage resulting from groundwater lowering.

7.9.3 Estimation of potential environmental effects

As is discussed in Chapter 15, in some circumstances, dewatering and groundwater control works may result in a risk of unacceptable environmental effects including, but not limited to, ground settlement, reduction in yield of nearby wells, and depletion of groundwater-dependent features. For dewatering systems of any significant scale, the potential for environmental effects, and the consequent need for mitigation measures, should be assessed during design. Assessment of environmental effects is discussed in Section 15.9.

7.10 NUMERICAL MODELING

Numerical modeling of groundwater flow problems has been carried out since the 1960s, but it is only since the 1980s and 1990s that advances in personal computer (PC) technology have made this approach viable for a wide range of groundwater lowering problems. Computers are now so ubiquitous in geotechnical engineering that not only is there a computer on every desk but they are also in many site engineer's field bag, and powerful modeling software is available at a cost similar to the price of this book! Numerical modeling as part of the design of dewatering and groundwater control systems is here to stay. However, it is important to identify when it can be used most effectively and to be aware of the potential pitfalls.

PCs can be used in design in two principal ways:

1. By use of spreadsheet programs to evaluate design equations that might previously have been carried out by hand. It is fairly easy to write routines for a spreadsheet to allow repeated sets of calculations to be performed as part of sensitivity or parametric analyses. Use of the PC dramatically speeds up this process compared with hand calculation.
2. By use of a numerical groundwater modeling package to solve complex groundwater flow problems that would not normally be amenable to solution by other means.

This section will concentrate on the use of numerical groundwater models applied to dewatering problems. It will describe the general approach that should be adopted and will not go into the details of the modeling process, which is described in texts such as the work of Anderson and Woessner (1992). Although numerical modeling packages are becoming easier to apply, it is vital that anyone contemplating their use understands the theoretical basis and limitations of the program in question. In complex or unusual situations, help should be obtained from an experienced groundwater modeler.

Essentially, a numerical model breaks down the overall problem, its geometry and boundary conditions, into a number of discrete smaller mathematical problems that can be solved individually. An iterative process is often carried out, whereby the solution to each smaller problem is adjusted until there is acceptable agreement at the boundaries between the smaller problems. It is sometimes stated that numerical models produce "approximate" solutions. This is true in the mathematical sense, because there will be a small difference between an analytical solution and the numerical results but, in an engineering sense, the numerical output is a pretty accurate reflection of the input data and the groundwater model that has been formulated.

The key point is that the numerical package is only following instructions given to it by the user. If there are errors in the input data or, more importantly, if the conceptual model (on which the groundwater model is based) is unrealistic, gross errors may result. It is an old cliché but the phrase "garbage in, garbage out"—meaning that the results can only be as good as the input and instructions—is very true for groundwater modeling.

The conceptual model (see Section 7.4) is the critical starting point for any modeling exercise. If the conceptual model is not a good match for actual conditions, then the output will be of questionable value.

It is important to select a numerical modeling package appropriate to the problem in hand. Some packages were originally developed for use in water resources modeling of large areas of aquifers for regional-scale studies. These packages can be useful for large dewatering works in highly permeable aquifers with large distances of influence, but may be less applicable for smaller-scale seepage problems. Another group of packages were developed for geotechnical problems such as seepage beneath cofferdams or through earth embankments; these may be appropriate for small-scale problems.

The modeling package used must be capable of solving the type of problem. Most packages can solve steady-state problems, but not all are designed for transient time-dependent seepage. Most groundwater flow problems are three-dimensional to some degree but, in some cases, it may be possible to simplify conditions to two-dimensional flow without significant error; some packages can model three-dimensional flow, but many are limited to two dimensions only.

The numerical modeling process can be divided into a number of stages:

1. Development of the conceptual model (see Section 7.4). Even though it does not involve touching a computer, this is the most important stage of the numerical modeling exercise. The conceptual model must quantify the geometry, aquifer parameters, and boundary conditions that will be used to define the numerical model. If the conceptual model does not reflect actual conditions, the numerical modeling results are unlikely to be realistic or useful.
2. Selection of software and setting up of numerical model. Once the conceptual model is defined, software capable of modeling those conditions can be selected. The software is then used to create the groundwater model (the set of instructions defining the relevant geometry, properties, and boundary conditions) for the problem. Any errors or omissions in the input data and instructions will affect the results produced by the model.
3. Verification and calibration. These activities are essential to allow any errors in the input data or model formulation to be identified and for the user to develop some degree of confidence in the validity of the output. The aim of verification is to answer the question: Has the model done what we intended it to do? To answer this question, the input data must be scrutinized for errors, and the output of the model must be compared with known analytical solutions. It is unlikely that an analytical solution will be available for the whole model, but it may be possible to simplify all or part of the model and compare it

with results calculated by, for example, flow net or equivalent well methods. If errors are detected, these are corrected and verification repeated until acceptable agreement is obtained. After verification, the model should be calibrated against field data, such as observation well readings in various parts of the modeled area. Calibration is a trial-and-error procedure whereby the model parameters and boundary conditions are varied (within realistic ranges chosen from the conceptual model) until there is acceptable agreement between the field data and the model output.

4. Prediction and refinement. A verified and calibrated model can be used to predict the results of interest (flow rates, drawdowns, settlements, etc.). Parametric and sensitivity analyses can be carried out to assess the effect on results of different well arrays or aquifer conditions. For larger or longer-term projects, it may be possible to refine predictions by further validation and calibration against results from monitoring of the dewatering system—this can be used as part of the observational method.

7.10.1 Potential applications of numerical modeling

There are a number of ways that numerical models can be used in the design of groundwater lowering systems. Perhaps the most obvious is to use the model, following a comprehensive site investigation, as a predictive design tool to finalize the dewatering system. The conceptual model is developed from the site investigation results, and the model is run repeatedly, adding, removing, or relocating wells and pumping capacity until the target drawdown is obtained at specified points, or other design requirements are satisfied.

There is another way that numerical modeling can be used, as an aid both to design and site investigation. If a groundwater model is created at an early stage of the project (perhaps using a conceptual model based on the site investigation desk study), it can be used as a preliminary design tool to crudely model the effect of possible ground conditions and construction options. Output from the model may highlight particular issues to be addressed by the ground investigation. Similarly, the effect of changing the size of excavation, depth of cutoff walls, and so on, can be investigated; this may be useful information for designers. Any potential side effects of groundwater lowering can also be quantified. The model is then developed, recalibrated, and refined as additional data are gathered and continue to provide information to designers on the effect of various options. This approach has been successfully adopted on a number of larger projects, including those in which the observational method was used.

Numerical groundwater models are not needed for every dewatering project. Numerical modeling is of most value in cases where traditional analytical solutions are either unavailable or too cumbersome for practical use. Situations where numerical models should be considered include

- *Complex geometry and geology*. Where the conceptual model or dewatering system is complex, perhaps with multiple aquifers and intermediate aquitards, nearby recharge or barrier boundaries, or artificial recharge systems, suitable analytical methods may not be available or their use may require such simplification of the conceptual model that the resultant errors or uncertainties are unacceptable. Numerical models have the potential to allow a design to represent the conceptual model with fewer simplifications, thereby giving more realistic and relevant results.
- *Anisotropy and nonuniform permeability conditions*. Analytical methods are available which can address anisotropic permeability and zones of different permeability, but these are applicable to only a narrow range of cases. Numerical models have the potential to allow the effect of these conditions to be assessed either as part of the main design process or as part of sensitivity or parametric studies. Bevan et al. (2010) describe an example in which numerical modeling was used to investigate the effect on a major dewatering system of large-scale variations in permeability in the area of the excavation.
- *Transient analyses*. Where nonsteady-state conditions, such as rate of drawdown or rate of water level recovery after the end of pumping, are of interest, numerical modeling can be very useful. This is particularly the case for multiple well systems with nonuniform geometries. Transient numerical models are often significantly more time-intensive and require better calibration data than steady-state models.
- *Environmental effects*. Numerical modeling is widely used on major dewatering projects to assess the potential environmental effects on the groundwater environment (see Section 15.9). Where the potential effects of concern include the movement of contaminated groundwater, it is essential that the modeling software used has the capability to simulate the transport of contamination in groundwater flow. Numerical modeling is typically used also to aid the design of mitigation measures. For example, where dewatering-related settlements are a concern, modeling scenarios may be run to assess the effectiveness of different locations and geometries of cutoff walls and artificial recharge wells. Crompton and Heathcote (1993) and Edwards (1997) describe a case in which numerical modeling was used to predict long-term groundwater level increases (and the corresponding requirement for long-term dewatering) when a tidal barrage was constructed across Cardiff Bay, United Kingdom.

- *Multiple scenarios*. A huge positive of numerical modeling is the ability to establish base numerical models and then vary key parameters or aspects, to allow multiple scenarios to be modeled. This can be of great value in helping the project team understand the risks if ground conditions or boundary conditions found in practice vary from the initial conceptual model.

7.10.2 Potential pitfalls of numerical modeling

Fundamentally, numerical modeling is no better or worse than analytical methods of dewatering design. Done well, by experienced design teams, both methods can give great results. Conversely, applied unwisely, both approaches can result in poor estimates of discharge flow rates and drawdowns, inappropriate designs, and consequent problems during construction.

With any method of design, it is essential that the outputs of the design process are validated or "challenged" to try and identify potential errors or unrealistic results. With numerical modeling, the review and validation of design outputs is especially important. Some of the potential pitfalls which can affect numerical modeling exercises are outlined below:

- *Treating the model as a "black box."* There is a real temptation, especially among those new to numerical modeling, to consider the modeling software packages, and the numerical models constructed from them as "black boxes," the workings of which are a mystery. This approach is a mistake. All software packages have their limitations and simplifications. The user must be aware of the limitations of the software to ensure that it is being applied appropriately. Similarly, because of software constraints, the models formed using the software may not be a perfect match for the conceptual model. It is essential that any simplifications in the numerical model are identified and understood.
- *Divergence between the conceptual and numerical model*. Any conceptual model will, unavoidably, be an imperfect representation of reality. Similarly, a numerical model will almost always involve some simplification of the conceptual model, to allow a workable model to be formulated. There is a risk, if the modeling specialist is not working closely with the dewatering designer or other team member who developed the conceptual model, that the numerical model will diverge unacceptably from reality. It is essential that numerical modeling is not carried out in isolation but is fully integrated with the rest of the dewatering design process.

- *Expecting a unique solution*. It is important that dewatering designers recognize that numerical models may, in some circumstances, give nonunique solutions. This can occur where complex geometries and multiple boundary conditions exist, and when the available calibration data might not allow the effect of a particular feature—such as a surface water body potentially linked to the aquifer—to be uniquely quantified. Such uncertainty of model outputs might require the design process to include sensitivity analysis and gathering of more data to allow more detailed calibration of the model.
- *Lack of empirical checks of modeling results*. Where a dewatering system is designed based on a numerical model, it is essential that empirical checks are carried out on the proposed dewatering system. For example, well yields, well spacing, and well depths should be compared with typical values given elsewhere in this book. If this is not done, then the resulting dewatering system may be unworkable.
- *Presenting a false sense of precision*. Most numerical modeling software packages provide results in the form of tabulated values of water levels and flow rates, which can be easily converted to graphs and contour plots that can then easily be incorporated in design reports. There is real risk that people reading the results will be impressed by the presentation of results and forget about the underlying simplifications and uncertainties. It is essential that those carrying out and reporting on numerical modeling work communicate the uncertainties and do not give a false sense of precision to the results. For example, care should be taken in selecting the number of significant figures used when quoting results, and it may be appropriate to quote key results (such as discharge flow rates) as a range of values rather than a single figure.

7.11 DESIGN EXAMPLES

Some design examples are presented in Appendix 4 to illustrate the application of the methods presented in this chapter. For ease of reference, the relevant equation and figure numbers are noted. The examples do not merely cover the numerical aspects of design, but also discuss some of the issues over which "engineering judgment" must be exercised.

Design example 1: Ring of relatively closely spaced deep wells in a confined aquifer. This case is modeled as an equivalent well using radial flow equations.

Design example 1a: This analyzes the case of example 1 using the alternative method of using shape factors for flow to equivalent wells in confined aquifers.

Design example 2: A line of partially penetrating wellpoints alongside a trench excavation in an unconfined aquifer. This case is analyzed using as an equivalent slot under plane flow conditions. The effects of assuming different aquifer depths and of including the contribution from radial flow to the end of the slot are assessed.

Design example 3: Ring of widely spaced deep wells around a large excavation in a confined aquifer. The cumulative drawdown method is used to design the system.

Chapter 8

Sump pumping

8.1 INTRODUCTION

Scott (1980) defines a sump as "a pit in which water collects before being bailed or pumped out. The pump suction dips into a sump."

By definition, a sump is at a low level in relation to surrounding ground surfaces so that any water will flow to it due to gravity. For construction projects, water is removed from sumps using suction or submersible pumps, not by bailing.

This chapter addresses the formation of pumping sumps and associated gravity drainage channels for the control of surface water and groundwater. Case histories are used to describe situations in which sump pumping was used appropriately and problems were created. The pumps suitable for use in sump pumping are dealt with in Chapter 13.

8.2 APPLICATIONS OF SUMP PUMPING

Sump pumping is the most basic of the dewatering methods. In essence, it involves allowing groundwater to seep into the excavation, collecting it in sumps, and then pumping it away for disposal. Sumps are provided for two separate purposes, although the form of a sump may be similar for either requirement:

1. To collect surface water runoff channeled to it using collector ditches or channels for discharge to a disposal point or area (see Section 5.2).
2. To collect and discharge pumped water due to lowering of the groundwater for a shallow excavation; also, sump pumping may be required for gravity drainage to toe drains of battered slopes (see Section 4.7).

The preferred method of disposal of water collected from the sumps is by pumping. Typically, each sump is equipped with a robust and simple pump—a "sump pump" (see Section 13.4). Sump pumping can be a very effective and economic method of achieving modest drawdowns in well-graded coarse soils (such as gravelly sands, sandy gravels, and coarse gravels) or in hard fissured rock.

Unfortunately, under some conditions, the use of sump pumping can lead to major problems. These problems arise primarily, because the flow of water into the excavation can have a destabilizing effect on fine-grained soils. This can lead to fine soil particles being washed from the soil with the water—this is known as "loss of fines." Loss of fines can lead to ground movements and settlements, because material is being removed from the soil, giving the potential for the formation and collapse of subsurface voids (see Section 15.4.3). Disposal of the water can also create problems, because if loss of fines occurs, the discharge water will have a high sediment load, which can cause environmental problems at the disposal point (see Section 15.8). These issues are discussed further in Sections 8.7 and 8.8.

Powers (1985) lists soil types in which the use of sump pumping has a significant risk of causing loss of fines. These include

1. Uniform fine sands
2. Soft noncohesive silts and soft clays
3. Soft rocks in which fissures can erode and enlarge due to high water velocities
4. Rocks in which fissures are filled with silt, sand, or soft clay, which may be eroded
5. Sandstone with uncemented layers that may be washed out

In these soil types, even the best engineered sump pumping systems may encounter problems. Potential problems can be avoided by using groundwater lowering using an array of wells (wellpoints, deep wells, or ejectors) with correctly designed and installed filters. Provided that the wells are located outside the main excavation area, these methods have the advantage of drawing water *away* from the excavation, improving stability and avoiding the destabilizing flows into the excavation that are associated with sump pumping.

8.3 SURFACE WATER RUNOFF

When an excavation exposes low-permeability soils, all surface water, whether derived from rainfall or from any other source, should be controlled so that any occurrence of ponding is prevented. Movement of construction plant through ponded surface water will lead to a deterioration of the surface, especially on clayey soils. This will inhibit the efficient use of the plant. It is good practice to form the surfaces of the construction areas so that they are not level; they should be gently sloped so that all surface water is shed to suitably sited and constructed collector channels or drains (see Section 5.2).

The fall on the bed of a collector drain should be sufficient to minimize silting up, but not so steep as to cause erosion. Near the sump, it may be prudent to increase the width of the drain to allow a flow velocity low enough to prevent erosion. The alternative is to provide check weirs at intervals along the line of the drain.

The surface water runoff within an excavation should be channeled to a conveniently located pumping sump, as shown in Figure 8.1. The forms of construction of the sump(s) are described below.

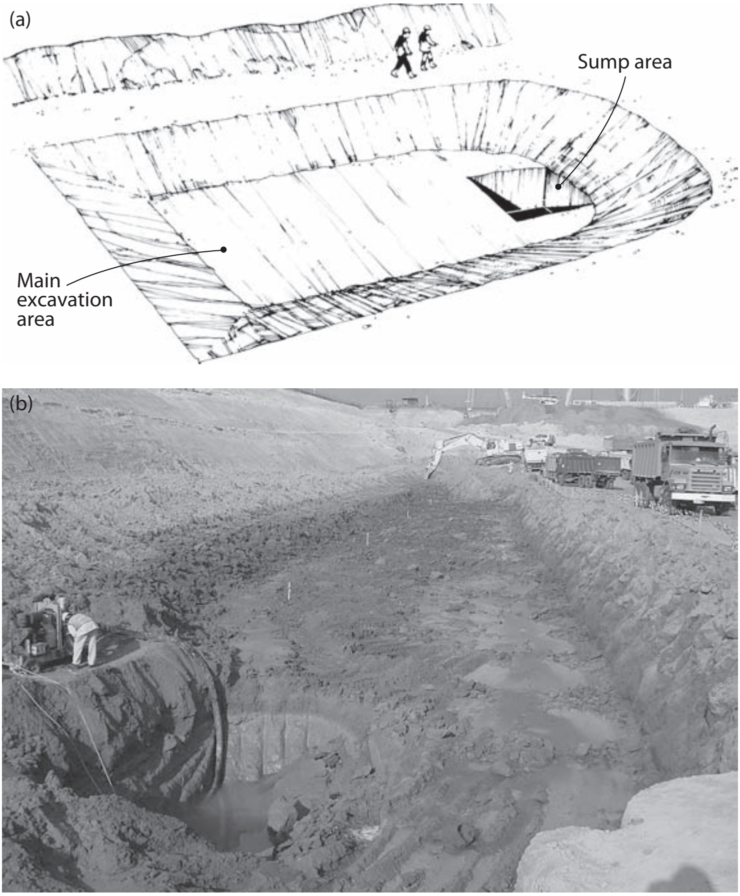


Figure 8.1 (a) Typical sump within the main excavation area. (From Somerville, S.H., Control of groundwater for temporary works. Construction Industry Research and Information Association, *CIRIA Report 113*, London, 1986. Reproduced by kind permission of CIRIA: www.ciria.org.) (b) Sump dug in the corner of the excavation. Without drainage ditches feeding water to the sump, water will pond at the base of the excavation, and a wet excavation results. (Courtesy of WJ Groundwater Limited, Bushey, U.K.)

8.4 PUMPING SUMPS

At all times, the depth of a sump must be a generous amount deeper than the bed of the collector drain(s) leading into it and of the formation level of the excavation. A sump should be substantially larger than that needed to accommodate the pump(s). Surface water flowing to the sump is likely to transport fines. These are likely to be abrasive and capable of causing wear and damage to the pumping equipment. A sump of a generous size will allow for some settlement of the larger (and probably more abrasive) fines. Adequate provisions should be made for periodic servicing of the pumps and removal of the accumulated sediment. The pump should be suspended so that the bottom of the unit is approximately 300 mm above the bottom of the sump to allow for some buildup of sediment. This does not apply to smaller, shallower sumps in which only the suction hose is in the sump.

A common arrangement for a sump is to suspend the pump in a 200-L drum or a similar container (Figure 8.2a) with many holes punched through the sides. An annulus of fine gravel, which acts as a crude filter, is placed outside the drum. Forms of sump construction are shown in Figure 8.2.

The sumps should be dug to a greater depth than the main excavation and should be maintained in their original form throughout the construction period, although it should be deepened, if necessary, as excavation proceeds. This will

1. Allow placement of filter media that may be necessary to minimize loss of ground
2. Keep groundwater below the excavation level at all stages of the work
3. Allow changes to be made in the construction scheme for the main excavation

Most sumps are formed by excavation, with the sides temporarily supported by sheeting for stability, before the body of the sump (i.e., the drum or similar) is placed. In certain circumstances, jetted sumps may be preferred (see Figure 8.3). A suitably sized placing tube is jetted into the ground to the required depth. A disposable intake strainer and flexible suction hose is lowered into the placing tube in a manner similar to the positioning of a disposable wellpoint and riser pipe. Filter media is placed within the placing tube, around the strainer and riser pipe as the placing tube is withdrawn. The upper end of the riser pipe is connected to a suitable pump.

Generally, the maximum effective depth to which a well-maintained, vacuum-assisted, self-priming centrifugal surface pump will operate is approximately 6 m below the top of the sump. For excavations of greater depth, it will be necessary to reinstall the pumps at a lower level or to use a suspended submersible pump, which can be lowered down a lined shaft or perforated steel tube.

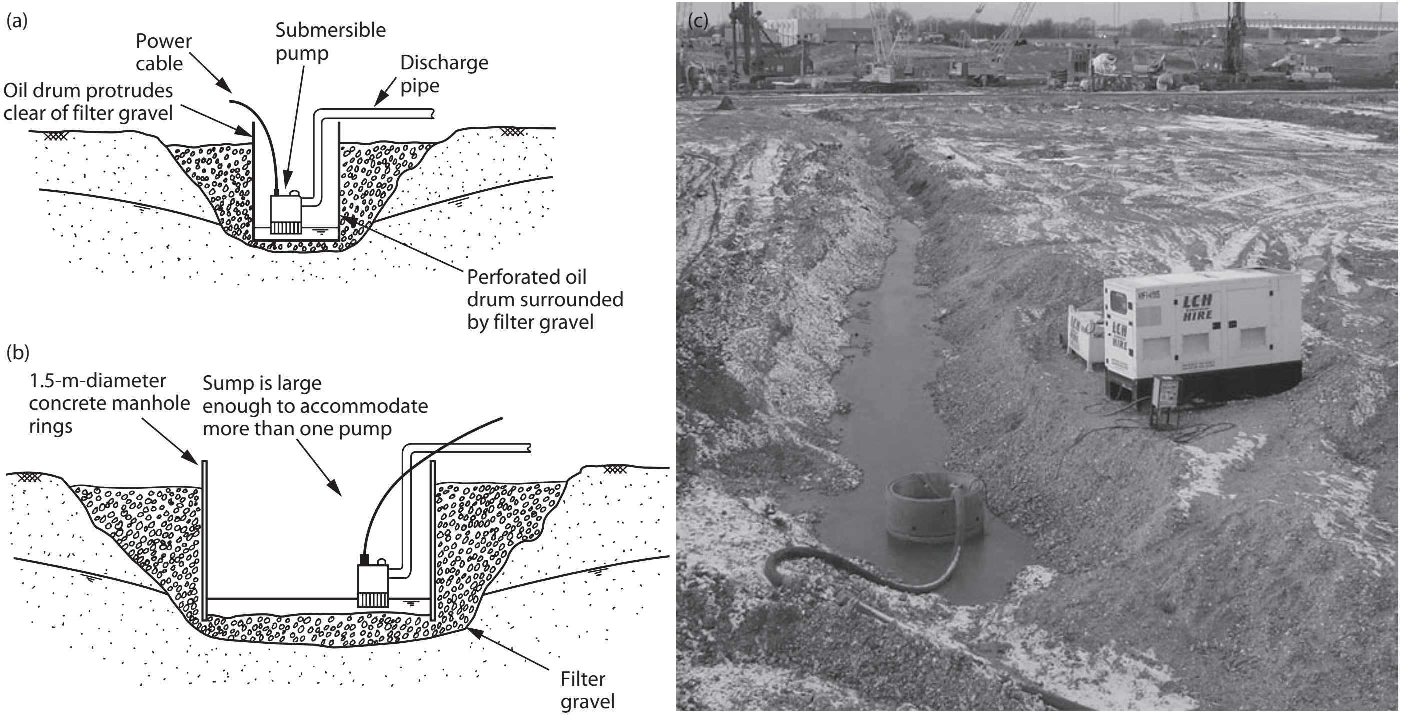


Figure 8.2 Typical forms of sump construction. (a) Small sump formed using perforated oil drums. (b) Large sump formed using concrete manhole rings. (c) Concrete manhole ring sump fed by a drainage ditch. (Courtesy of WJ Groundwater Limited, Bushey, U.K.)

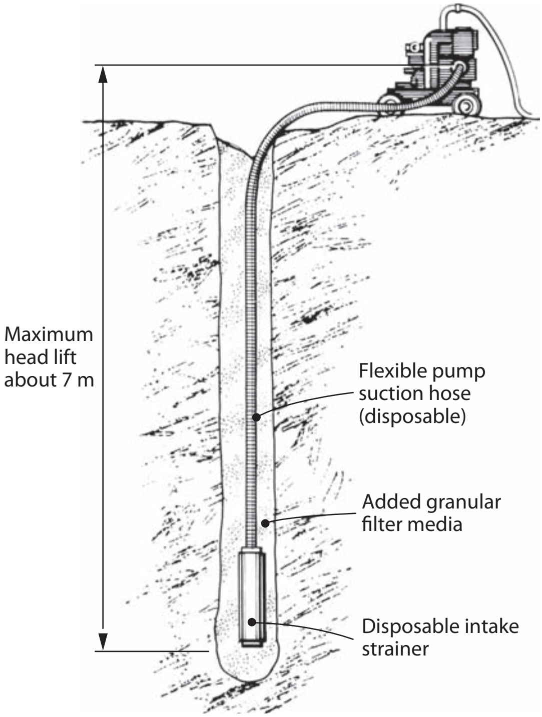


Figure 8.3 A jetted sump. (From Somerville, S.H., Control of groundwater for temporary works. Construction Industry Research and Information Association, *CIRIA Report 113*, London, 1986. Reproduced by kind permission of CIRIA: www.ciria.org.)

The need for sufficient pumping capacity is paramount, because a greater pumping capacity is needed to initially dewater an excavation than is required to maintain the water level at a steady state in its final, lowered position. The pumping plant should be installed in multiple units so that additional units required to provide increased capacity for the initial pumping load can be shut down as the required levels are reached. The spare pumpsets should be left in position to act as standby pumps in case of breakdown or other emergencies. Intermediate pumping may also be required in a subsequent backfilling state.

8.5 DRAINAGE OF SIDE SLOPES OF AN EXCAVATION

Within an area of excavation, toe drains and pumped sumps may be required to collect seepages of groundwater from the side slopes as well as rainfall runoff.

The collection of seepages from side slopes has been addressed in Section 4.7. The provision of a toe filter drain (Figure 4.8) is important. The water pumped from the toe drainage system should be clear (i.e., no fines). The presence of fines in the pumped seepage water would indicate that the filters are inadequate, fines are being continually removed, and, eventually, the slope will become unstable. The sumps of the seepage collection system should be as described previously in Section 8.4. The expected seepage rates of flow should indicate the dimensions of the sumps that will be required.

8.6 SUMP PUMPING OF SMALL EXCAVATIONS

Initially, the discharge water from any pumping operation will be discolored due to the presence of fines. Continuation of the discoloration of the water can be tolerated only if the pumped water is entirely derived from surface runoff. It cannot be tolerated if the discoloration continues when the flow is derived from groundwater; this is a warning of potential danger, because it indicates continuing withdrawal of fines from the formation. This dangerous condition is often seen in small excavations and trenches of modest depths.

A considerable amount of excavations are carried out for trenching (see the work of Irvine and Smith 2001). For these, sump pumping can be used safely in permeable soils such as gravels and clean sand/gravel mixtures. It is simple and economical and is particularly appropriate to use in trench-sheeted excavations; the sheeting tends to limit the inflow to be pumped (see Figure 8.4), provided that the discharge water is clear. When dealing

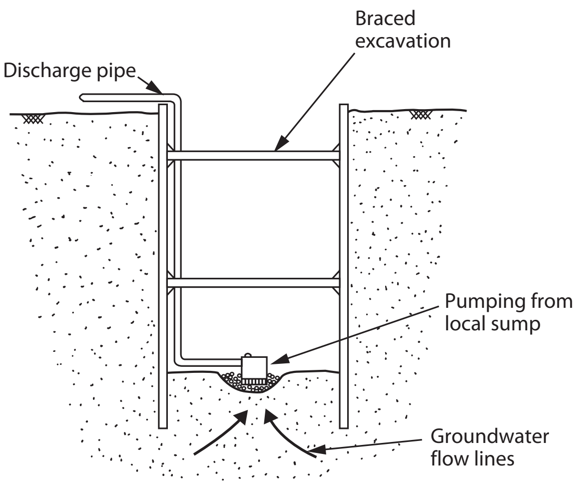


Figure 8.4 Sump pumping from within the trench.

with finer grained soils (such as silty sand), a system of wellpoints adjacent to the side(s) of the trench should be used instead of sump pumping.

8.7 SUMP PUMPING PROBLEMS

In practice, sometimes a wellpoint or deep well installation fails to establish the total lowering required, perhaps because the wellpoints were not placed deep enough or were spaced too far apart. In such circumstances, the safe, correct procedure is to install a second stage of wellpoints (see Section 9.10).

There have been occasions where this correct procedure was not followed, but instead, efforts were made to achieve that final lowering, for example, another 0.5 m, by additional sump pumping. Figure 8.5 shows a typical result of the messy conditions created by this incorrect application. In this case, the foundation slab for a valve chamber was required to be constructed at a modest depth below the water table. The proposed excavation into a very silty sand stratum was ringed by a wellpoint installation—the correct approach. Unfortunately, the requisite amount of lowering was not achieved. The reason for this shortfall in lowering

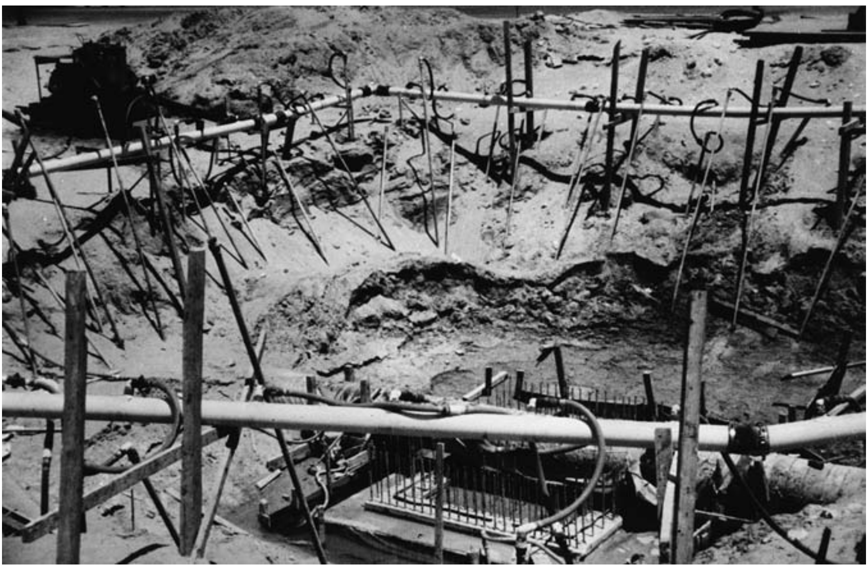


Figure 8.5 Significant ground movement caused by inappropriate sump pumping. The wellpoint risers around the perimeter of the excavation had originally been installed vertically. Loss of fines due to poorly controlled sump pumping resulted in ground movements, distorting the wellpoints from the vertical.

is not known. Perhaps, the wellpoints were not installed deep enough, there were many air leaks in the pipework, and therefore, a good vacuum was not established, or the permeability of the soil was less than predicted, and therefore, the potential achievable lowering was lower. There seems to have been no close observation of what was happening while sump pumping was continued. Sump pumping resulted in the continuous removal of fines from the sides of the excavation. It can be seen that the wellpoints and their risers (originally vertical) were moved toward the sump-pumped excavation by the considerable movement of the surrounding ground.

Another example of an effect of sump pumping is shown in Figure 8.6. A bund of predominantly granular material was placed to protect an excavation sited beside a tidal estuary. A cutoff wall was formed to penetrate an underlying stratum of low permeability and so exclude water in the estuary as the tide level rose. Before the cutoff was closed, it was judged to be acceptable to carry out limited sump pumping of the excavation so that work inside the bund could be continuous even around the

Figure 8.6 Outwash fans due to sump pumping. The suction hoses for the sump pumps were mounted on the crude pontoon shown on the left of the photograph. This helped pumping continue as the water level within the excavation varied with the tide.

times of high tides. There came a day when pumping was accidentally allowed to continue for longer than necessary. The photograph shows a series of outwash fans due to the transport of fine sand in prolonged seepage flows.

If the transport of fines cannot be controlled by the construction of sumps with adequate filters and if the discharge water cannot be adequately treated, a change in the dewatering method used should be considered. The use of a well system (wellpoints, deep wells, or ejectors) with adequate filters is normally a viable alternative to prevent loss of fines from occurring.

8.8 DISPOSAL OF WATER FROM SUMP PUMPING OPERATIONS

If the water flowing to the sumps is removing fines from the soil, the pumped water will have a significant sediment load of sand, silt, and clay-sized particles. Sump pumps are normally tolerant of some sediment in the water and are likely to continue to operate, unless the sediment load is exceptionally high, at which point they can become choked with sand and silt. However, the discharge of the sediment-laden water at the disposal point is likely to cause environmental problems.

Increasingly, there are situations where sump pumping may be technically feasible, but where environmental concerns associated with the discharge of “dirty” water from the site preclude the use of sump pumping. Discharge of water with a significant sediment load can cause a range of potential problems (see Section 15.8 for a more detailed discussion of this issue). If the water is discharged to a sewer, the sediment may build up in the sewer, reducing capacity and causing the sewer to back up. If the water is discharged to a watercourse, the sediment will have a harmful effect on aquatic plant, fish, and insect life.

It can be difficult to economically remove silt and clay-sized particles from discharge water. However, in recent years, the use of mobile “silt traps” based on the principle of lamella plate settlement has become common; when appropriately deployed, these units can significantly reduce the sediment load in discharge water. Water treatment options for suspended solids are discussed in Section 15.8.2.

The use of water treatment to remove suspended solids from water derived from sump pumps should only be considered where investigations have confirmed that the loss of fines from the soil (see Section 8.2) will not cause the creation of voids or localized instability in the excavation. If any risk exists of instability caused by loss of fines, then sump pumping should be curtailed and replaced with an alternative method such as the use of a well system (wellpoints, deep wells, or ejectors) with adequate filters, where loss of fines should not occur.

8.9 CASE HISTORY: SUMP PUMPING OF LARGE EXCAVATION

Quite large-scale projects have been dealt with adequately by sump pumping of relatively high-permeability soils, such as alluvial gravels with some sand and having little or no "fines."

Morrison Construction Limited formed an open reservoir upstream of the city of Aberdeen beside the River Dee, to the instructions of Mott MacDonald, the engineer appointed by the client, Grampian Regional Council Water Services. The water surface area of the reservoir was on the order of 8 ha, with a storage capacity of 240,000 m^3. The reservoir was created by forming a horseshoe-shaped containing embankment to marry with the existing flood embankment of the River Dee and all to similar height; the length of the reservoir embankment constructed by Morrison Construction Limited was 1150 m. The flood plain soils beneath the reservoir are alluvial sandy gravels of very high permeability: one pumping test indicated a permeability on the order of 2.5×10^{-2} m/s. The levels of the groundwater were substantially influenced by the river level, not surprising in view of the very high permeability of the flood plain deposits.

The engineer required the formation level of the floor of the reservoir to be some 1.5 m below the normal river level. The floor and the sides of the reservoir were required to be lined with an impermeable membrane of low-density polyethylene sheeting laid on a 50-mm-thick sand bedding. Thus, to lay the impermeable membrane and satisfactorily joint adjacent sheets, it was necessary to lower the groundwater level to some 2 m below the normal river level.

A large central pumping sump (Figure 8.7) was excavated to a depth of about 4 m below the level of the underside of the membrane. Three drainage trenches (approximately 80–120 m long) were excavated to about 2 m depth below the level of the underside of the impermeable membrane, sited to radiate out from the central sump to the riverside perimeter of the reservoir (Figure 8.8).

Into these drainage trenches 150-mm perforated unplasticized polyvinyl chloride (UPVC), drainage pipes were placed, and the trenches were backfilled with 75-mm cobble-sized material. This facilitated the general drawdown of the groundwater beneath the area of the reservoir by pumping from one central sump; although there was some additional pumping toward the end of the membrane laying from an auxiliary sump sited close to the inside toe of the embankment.

The discharge from two 200-mm vacuum-assisted self-priming centrifugal pumpsets plus three 150-mm pumpsets, approximately 1000 m^3/h, was discharged to the River Dee. This sump pumping installation enabled Morrison Construction Limited to lay the specified sand bedding and membrane satisfactorily.

Figure 8.7 Dee Reservoir, central pumping sump. The sump pumps are composed of one electrically powered unit, supplied from a portable generator, and two diesel-powered units. (Courtesy of the Grampian Regional Council, Aberdeen, U.K.)

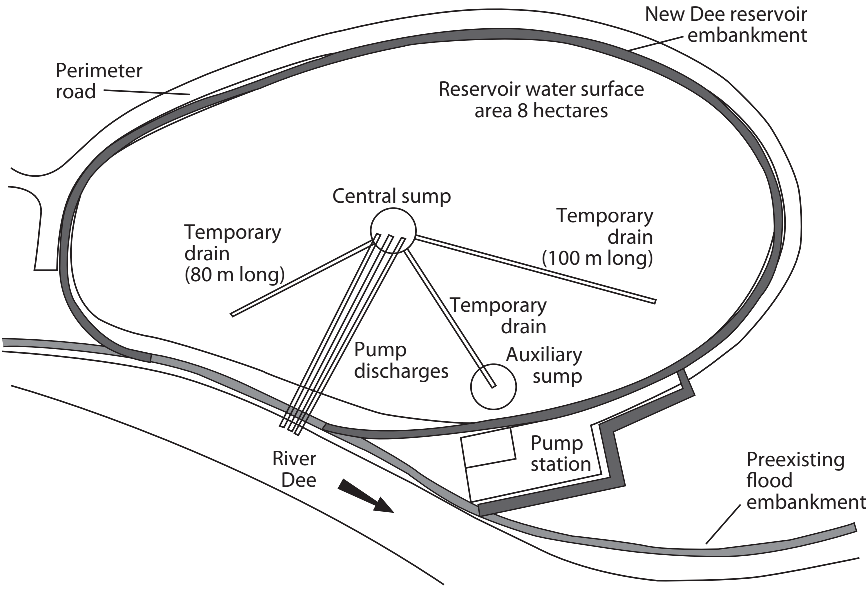


Figure 8.8 Dee Reservoir, plan showing positions of sump and temporary drainage trenches. (Courtesy of the Grampian Regional Council, Aberdeen, U.K.)

Chapter 9

Wellpoint systems

9.1 INTRODUCTION

For small- and medium-sized construction projects of limited depth, wellpointing is the most frequently used pumping method for the control of groundwater. It is frequently used for shallow pipe trenching and the like. The deep well system is usually more appropriate for deep excavations and is addressed in Chapter 10.

This chapter describes the wellpoint pumping method. Current good practices, installation procedures and practical uses and limitations are discussed. Variations to the wellpoint system to cope with differing soil conditions are also considered.

A variation on the conventional wellpoint system, applicable to pipeline trenching in open country and often referred to as horizontal wellpointing, is addressed in Chapter 11, together with other less commonly used groundwater lowering techniques including ejectors.

A wellpointing case history is described at the end of the chapter. It illustrates the success that can be achieved in dealing with difficult soil conditions by using adequate "sanding-in" procedures to provide satisfactory vertical downward drainage and so achieve acceptable pore water pressure reductions.

9.2 WHICH SYSTEM: WELLPOINTS OR DEEP WELLS?

There is a certain amount of confusion, probably due to some looseness of terminology, concerning the precise differences between wellpoint and deep well systems. Essentially, a wellpoint pumping system sucks groundwater up from a group of wellpoints to the intake of a pump unit and then pumps it to a disposal area. Its application is constrained by the physical limits of suction lift. In contrast, a deep well system consists of a group of wells, each having a submersible pump near the bottom of the well, which likewise pump to a disposal area; the method is not constrained by suction lift limitations. This is the key practical distinction between the two systems.

From consideration of both economic and technical criteria, it is likely that wellpointing will not be the most appropriate technique to use if the following conditions exist:

- Large excavations or where excavation depths are greater than 12 to 15 m.
- Where there is a pressure head in a confined aquifer below an excavation which should be reduced to preserve stability at the formation level.

For such conditions, a deep well system, sometimes supplemented by a system of relief wells, might be considered.

9.3 WHAT IS A WELLPOINT SYSTEM?

A wellpoint system consists essentially of a series of closely spaced small diameter water abstraction points connected through a manifold, to the suction side of a suitable pump.

The wellpoint technique is the pumping system most often used for modest depth excavations, especially for trenching excavations and the like (see Figure 9.1). In the appropriate ground conditions, a wellpoint system can be installed speedily and made operational rapidly. The level of expertise needed to install and operate a wellpoint system is not greatly sophisticated and can be readily acquired. However, as with any ground engineering

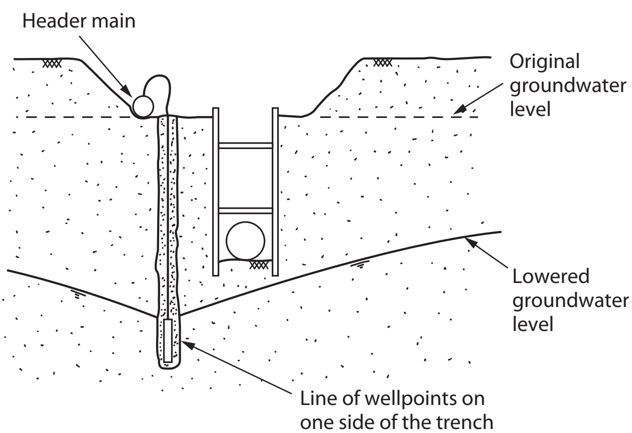


Figure 9.1 Single-sided wellpoint system. The section shows a trench with a line of wellpoints close to one side of trench.

process, having experienced personnel to plan and supervise the works can be crucial in identifying and dealing with any change in expected ground conditions.

A wellpoint is a small diameter water abstraction point (the well screen), sometimes referred to as a “strainer” (so called because of the wire mesh or other strainer of the self-jetting wellpoint) through which the groundwater passes to enter the wellpoint. They are installed into the ground at close centers to form a line alongside (Figure 9.1), or a ring around (Figure 9.2), an excavation. The perforated wellpoint is typically approximately 0.7 to 1.0 m in length and 40 to 50 mm nominal diameter. Each is secured to the bottom end of an unperforated pipe (the riser pipe) of slightly smaller diameter (38 mm diameter pipe is commonly used). However, where the “wetted” depth is limited due to the proximity of an impermeable surface (see Figure 9.13b), it is preferable that the length of the wellpoint should be shorter (usually 0.3–0.5 m) to restrict the risk of air intake at maximum drawdown. Often it will be necessary to install wellpoints at closer centers to compensate for the lesser screened length of each short wellpoint (see Section 9.8).

Each wellpoint is connected to a header main (typically of 150 mm diameter) that is placed under vacuum by a wellpoint pump (Figure 9.2). The header main is normally made of high-impact plastic, although steel pipe is

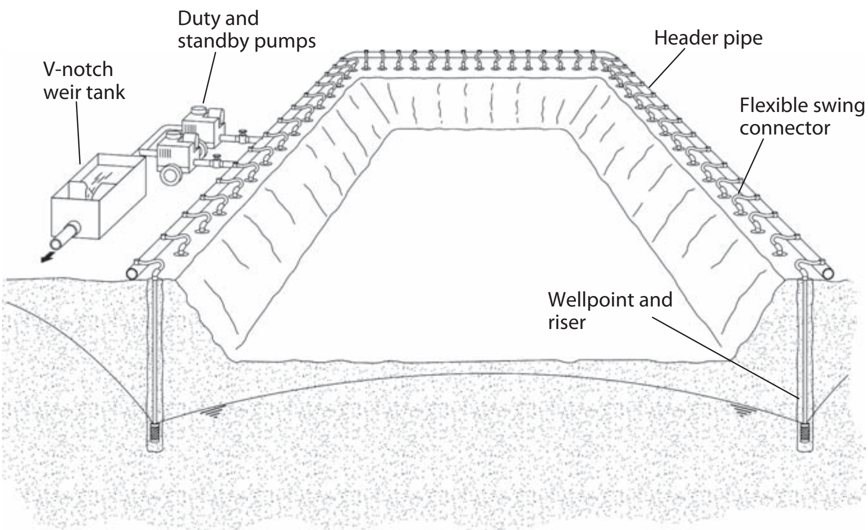


Figure 9.2 Wellpoint system components. (From Preene, M. et al., Groundwater control-design and practice. Construction Industry Research and Information Association, *CIRIA Report C515*, London, 2000. Reproduced by kind permission of CIRIA: www.ciria.org.)

sometimes used, especially when there is a risk of damage from construction activities. The pipe is typically supplied in 6 m lengths and is joined on-site by simple couplings that allow a certain degree of skew to allow the header main to be laid around gentle curves if necessary. For sharper curves, 90° and 45° bends are available, as well as tee pieces, blanking ends, etc.

The header main has connections for wellpoints at regular intervals, perhaps every meter. The individual wellpoint riser pipes are connected to the header main via "swing connectors" sometimes known simply as "swings." The swing connectors provide some flexibility in connecting the wellpoints (which are unlikely to be installed in a precise straight line) to the relatively rigid header main. Before the advent of readily available plastic hose, the connectors were made from a series of metal pipework bends which, when swung around, gave the necessary articulation—hence swing connectors. Nowadays swings are typically formed from flexible plastic hose (of 32–50 mm diameter), often with each swing incorporating a trim valve (Figure 9.3) to allow the abstraction rate of each wellpoint to be regulated if necessary (see Section 9.7).

The applied vacuum from the pump sucks the groundwater from the surrounding ground through the wellpoint screens, into the riser pipes, through the swing connectors and from there up into the header main and so to the pump intake. The pump then forces the water through the discharge main for ultimate disposal. Thereby the system causes local lowering of the groundwater within the area that it encompasses. The header main is effectively used as a manifold to allow a small number of wellpoint pumps to be connected to a large number of wellpoints (Figure 9.4).

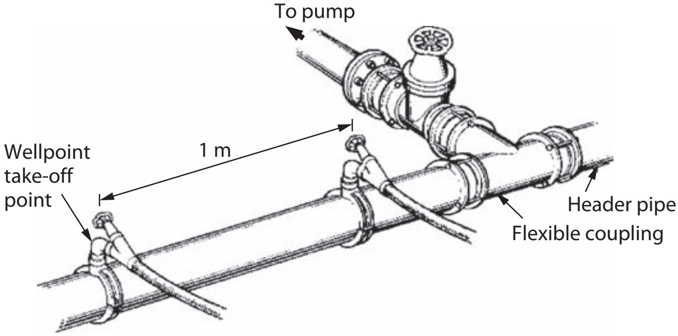


Figure 9.3 Flexible connection from wellpoint riser to suction manifold via trim valve. (From Somerville, S. H., Control of groundwater for temporary works. Construction Industry Research and Information Association, *CIRIA Report 113*, London, 1986: Reproduced by kind permission of CIRIA: www.ciria.org.)

Figure 9.4 Wellpoint pumps connected to header main. The set of three wellpoint pumps is connected to the header main, with valves to isolate each pump. The center pump is a standby pump to be used if either of the other pumps is out of service. (Courtesy of WJ Groundwater Limited, Bushey, U.K.)

Ideally, the header main should be just above the static groundwater level to minimize the amount of suction lift. This may entail excavating down to groundwater level before the installation of the wellpoints, followed by installation of wellpoints and header main at that level (Figure 9.5).

Ideally, the suction intake of a wellpoint pump should be at the same level as the header main. Often this will require that a small pit be dug to lower the pump body to the requisite level.

The amount of lowering that can be achieved by a wellpoint (or shallow well) system is limited by the physical constraints of a suction lift. It is generally in the range of 4.5 to 6 m, although a 7 m drawdown is not unknown. It depends substantially on the efficiency of the water/air separation device on the pump and the vacuum efficiency of the total installation (i.e., the airtightness of the system pipework). In addition, it depends on the structure and permeability of the soil mass. The width of the excavation is also pertinent to achieving the required amount of lowering at its center.

Where the land area available for construction is not constrained, the wellpoint system can be used for deeper excavations by means of a multistage wellpoint installation (Figure 9.6).

Figure 9.5 Wellpoint header main installed below the original ground level. Before dewatering, local excavations were made down to the natural groundwater level along the line of wellpoints, and the header main and pumps were installed at that level. (Courtesy of WJ Groundwater Limited, Bushey, U.K.)

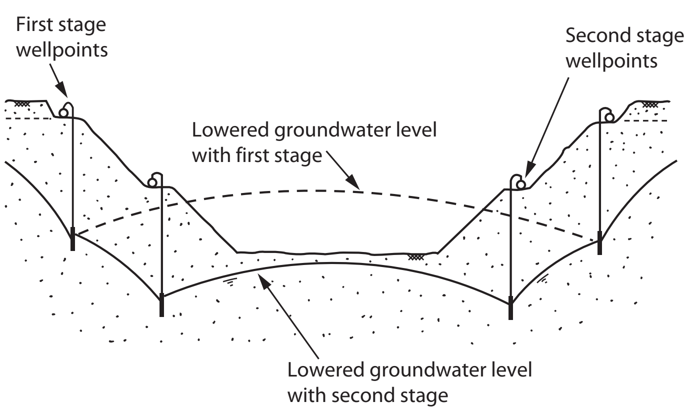


Figure 9.6 Two-stage lowering using wellpoints.

9.3.1 Types of wellpoint

The wellpoints are vital components of every installation. There are two types of wellpoints:

1. The self-jetting wellpoint (Figure 9.7a)
   These are known as self-jetting because they can be installed without the use of a placing tube. The wellpoint and riser are metal and therefore are rigid, with the wellpoints connected to the bottom of the riser pipe. This type may be recoverable for subsequent reuse.
   The wellpoint and riser are installed using high-pressure water supplied to the top of the riser pipe from a clean water jetting pump. There is a hollow jetting shoe below the wellpoint screen. Near the lower end of the shoe, a horizontal pin is located; above the pin is a lightweight loose-fitting ball. When the water pressure is applied to install the wellpoint, the ball is displaced downwards to allow the passage of the high-pressure water flow but the pin retains the ball within the jetting shoe. When pumping starts, the applied vacuum sucks the ball up onto a shaped spherical seating and thereby seals the lower end of the wellpoint, so that water from the surrounding ground can only enter through the screen section. The unperforated riser should extend to near the lower end of the wellpoint screen to minimize the potential for air intake at maximum drawdown.

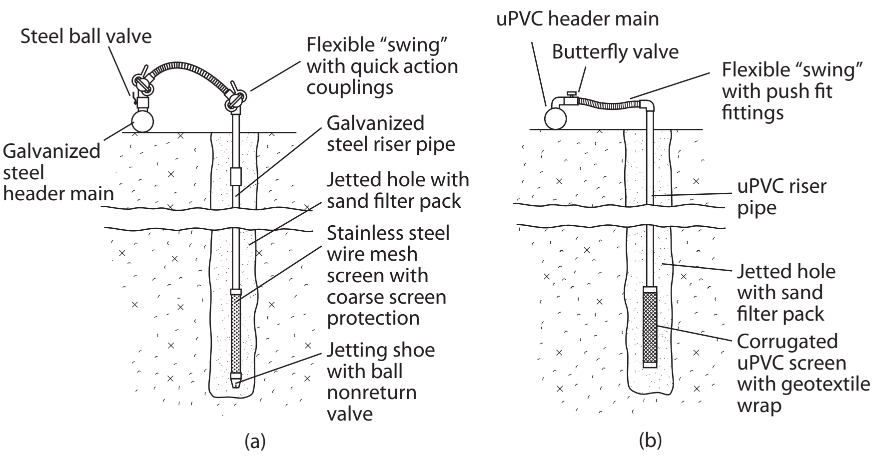


Figure 9.7 Disposable and reusable self-jetting wellpoints. Self-jetting wellpoint (a), disposable wellpoint (b). (From Preene, M. et al., Groundwater control-design and practice. Construction Industry Research and Information Association, *CIRIA Report C515*, London, 2000. Reproduced by kind permission of CIRIA: www.ciria.org.)

Although the self-jetting wellpoint can be subsequently extracted for reuse, it is not uncommon for the riser pipes to be damaged during extraction and so need straightening or even replacement before reuse. In addition, the wellpoint screens may need "desanding" before being suitable for reuse.

2. The disposable wellpoint (Figure 9.7b)
   The wellpoint and riser are usually made of plastic material and therefore inert to corrosion. They are installed using a placing tube or holepuncher using a similar high-pressure water jetting technique for installation as for the self-jetting wellpoint.

   Although many purpose-designed plastic wellpoints have been marketed for some time, current practice is to adapt low-cost, thin-walled convoluted uPVC perforated land drainage pipes to form disposable wellpoints with woven mesh stocking or "coco" wrapping. The latter forms a good filter and, in conjunction with normal sanding-in, is very effective even in difficult, silty soils. As with the self-jetting wellpoint, the riser (typically made of uPVC ducting) should extend to near the bottom of the screened length. The bottom end of the disposable wellpoint is sealed. Generally, the riser pipe is nominally 6 m long, but it can be longer.

   The disposable wellpoints are not recoverable but some of the plastic riser pipes can sometimes be recovered for future reuse. The disposable wellpoint is very appropriate to use for long-duration pumping duty.

9.4 WELLPOINT INSTALLATION TECHNIQUES

Self-jetting wellpoints and placing tubes used with disposable wellpoints are installed using high-pressure water supplied from a high-pressure jetting pump.

The jetting pump (see Section 13.3) most commonly used supplies water via the jetting hoses at a rate of approximately 20 L/s and at a pressure of 6 to 8 bar. The flexible jetting hoses are usually standard 63 mm fire hoses with instantaneous male and female connections. There are other more powerful jetting pumps and hoses that are often used for the holepuncher, pile jetting, and other similar heavy-duty uses.

The procurement of an adequate supply of jetting water for installation must be resolved for each site. A continuous supply of clean water for jetting is essential for efficient placing of all types of wellpoints, whichever installation technique is used. If the source of jetting water is restricted, consideration must be given to obtaining water by pumping from the first few wellpoints as each is installed.

Generally, the volume of jetting water required for installation per wellpoint with 6-m-long riser will be of the order of 1 to 1.5 m^3, but will depend greatly on soil conditions. However, it can be in the range 0.5 to 35 m^3, the latter figure being applicable to jetting in a very permeable river gravel.

Jetting in compact sands and gravels, and especially in open gravel, may be difficult and slow, mainly because the displaced or slurrified soil particles tend not to be washed from the jetted hole to the ground surface due to rapid dissipation of pressurized water into the open permeable formation. However, there is some compensation because in such permeable soils, sanding-in is unlikely to be required.

The wellpoint system is very flexible. For instance, after the initial installation, extra wellpoints can be placed speedily to deal with localized trouble spots. It is especially useful for shallow-depth trench excavations in which the pumping period is very often expected to be of short duration.

9.4.1 Installation of self-jetting wellpoints

The wellpoint and its riser pipe are assembled to the required length and a flexible jetting hose is attached to the top of the riser pipe via a jetting adapter. The other end of the flexible jetting hose is connected to the supply outlet from the high-pressure jetting pump.

It is prudent to form a small starter hole into which the wellpoint is positioned; or, for a line of closely spaced wellpoints to form a trench and so restrict general flooding of the site area. Before turning on the supply of jetting water, at the required wellpoint position, the jetting crew must upend the wellpoint and riser with jetting adapter and jetting hose all connected (Figure 9.8). There is considerable weight in the riser and hoses—this is a strenuous and tricky task.

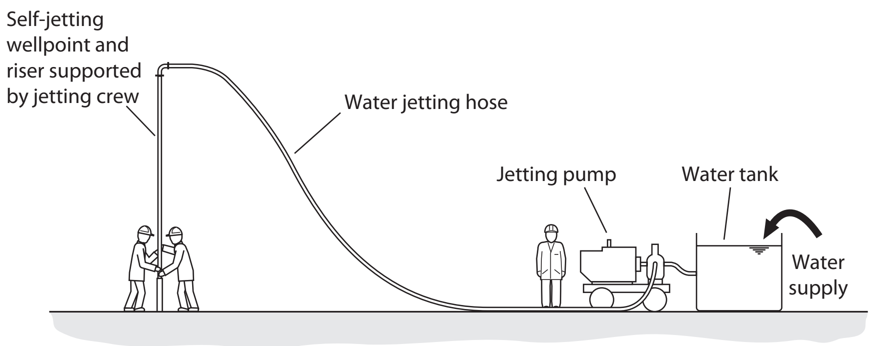


Figure 9.8 Installation of self-jetting wellpoint. (From Preene, M. et al., Groundwater control-design and practice, Construction Industry Research and Information Association, *CIRIA Report C515*, London, 2000. Reproduced by kind permission of CIRIA: www.ciria.org.)

The jetting pump forces high-pressure water down the metal riser pipe, past the ball at the lower end of the wellpoint. A strong jet of water emerges from the base of the wellpoint. The high-pressure water slurrifies the granular soil immediately below the bottom of the wellpoint, enabling downward penetration to be made until the required depth is achieved. The slurrified soil is washed to the surface with the jetting water and emerges in the annulus around the riser pipe. This turbulent flow of water and soil is colloquially known as the "boil." On reaching the required depth, the output from the jetting pump is throttled back during sanding-in. Then, and only then, should the water be turned off completely and the jetting adapter and hose disconnected.

In suitable soils, such as clean sands, the installation of self-jetting wellpoints is speedy—in some cases, it may be only a matter of a few minutes per wellpoint for the actual jetting to depth. In such circumstances, it is, therefore, very cost-effective.

As a rough guide, where the standard penetration test (SPT) N values are less than about $N = 25–30$, both self-jetting wellpoints and lightweight, easily manageable, placing tubes (for installing disposable wellpoints) can be used. Either technique requires minimal mechanical plant involvement but have greater need for manual labor.

In denser gravels, one of the controlling factors is the sand content of the soil. If the soil is sufficiently sandy to allow the jetting water to return to the surface as a boil, jetting will probably be feasible. If the gravels have little sand content and allow the jetting water to dissipate, the boil will be lost, and jetting will be difficult if not impossible.

9.4.2 Installation of disposable wellpoints by placing tube

For the installation of disposable wellpoints, a similar process is mobilized using a placing tube. High-pressure water from the jetting pump is applied to the placing tube via the appropriate fitting at the top of the tube (Figure 9.9). The high-pressure water emerges from the bottom of the placing tube and slurrifies the soil so that penetration to the required depth is achieved. The boil emerges around the outside of the placing tube. The placing tube is the temporary casing into which the disposable wellpoint and riser pipe are then installed centrally and sanded-in.

Modest cranage will be needed to handle the placing tube (e.g., a backhoe loader or a small 360° hydraulic excavator). However, if access for plant is restricted, where jetting conditions are easy, a lightweight 100 mm placing tube that can be manhandled may be used.

The placing tube itself is a robust open-ended jetting tube of a nominal 100 or 150 mm casing with a wall thickness of 4 to 5 mm. The fittings at the top of the tube are arranged such that not only high-pressure water can be applied

Figure 9.9 Installation of wellpoints using placing tube. (a) Steel placing tube is suspended from the excavator; (b) placing tube is jetted into the ground.

Figure 9.9 (*Continued*) (c) Plastic disposable wellpoint is installed. (Courtesy of Dewatering Services Limited, Sandbach, U.K.)

from a jetting pump but there is, in addition, the facility to apply compressed air to assist downward penetration through difficult ground conditions.

When an excavator is used to handle the placing tube, the top of the tube should be fitted with a platform or anvil such that if difficult ground conditions are encountered (e.g., a thin layer of stiff clay or random cobbles), the bucket of the machine can be used to press down on the platform when installation progress is slow. The anvil also serves to protect the cap and various fittings at the top of the tube.

On reaching the required depth, the jetting water is turned off, the cap at the top of the tube is removed, the wellpoint and its riser inserted to depth centrally inside the placing tube, and the placing tube is withdrawn as sanding-in progresses.

As a rough guide, the placing tube technique for wellpoint installation is appropriate where the SPT N values are about $N = 40$ or lower. Where many random cobbles are expected and the SPT values are about $N = 35–40$, and where SPT values are higher than $N = 40$, the holepuncher technique should be considered.

In recent years, there has been a tendency to extend the use of the placing tube technique to replace the more labor-intensive self-jetting wellpoint and lightweight placing tube techniques.

9.4.3 Installation using the holepuncher and heavy-duty placing tube

The holepuncher technique has been used successfully to progress through soils having SPT N values below $N = 30$ and up to about $N = 65$. Although it is often used to install wellpoints in hard or difficult ground conditions, it is also used for the installation of deep wells. For wellpoint applications, the holepuncher is typically suitable for installations up to 15 m deep.

The holepuncher (often called a "sputnik") is a simple robust form of wash boring equipment with the addition of a drop hammer driving facility. It consists of an outer heavy-duty casing (usually 200–250 mm nominal bore but can be up to 450 mm or even 600 mm for deep well installations) and an inner wash pipe with a weighted head which can also be used as a drop hammer. Usually, the top of the wash pipe has two intake ports for the supply of high-pressure jetting water and also has a facility for compressed air supply to be connected. The boil of washings rises to ground level in the annulus between the inner wash pipe and the outer casing (Figure 9.10).

Cranage will be needed for the use of a holepuncher. The crane must have free fall on two hoist lines. If compressed air is used to assist installation, the minimum compressor capacity needed is of the order of 55 L/s at about 10 bars.

The forerunner to the holepuncher was the heavy-duty placing tube, which is still used by some organizations. This equipment similarly consists of a thick-walled casing (usually 150–250 mm bore) with a box at the head containing lead weights. There are two jetting water connections and a compressed air connection—again similar to the holepuncher—provided just below the top. The inner jetting tube and lead-weighted head cannot be lifted independently of the outer casing, unlike the holepuncher. Hence, the displaced soil spoil is backwashed to ground level outside the casing tube. Similarly, a crane with free fall hoist lines is required to handle the heavy-duty placing tube. The wellpoint and its riser pipe are installed and sanded-in using the same procedures as for the placing tube method.

There are particular safety hazards associated with using a holepuncher or heavy-duty placing tube. The tube itself is a heavy and unwieldy device, not easily operated by crane drivers who have never used one before. The volume of jetting water and compressed air applied inputs a lot of energy into the ground; this can result in gravel and cobble fragments being ejected from the top of the tube with great velocity, to land some distance away. Especially in urban areas or in confined sites, the risks to operatives and the public should be carefully assessed when considering using the holepuncher technique.

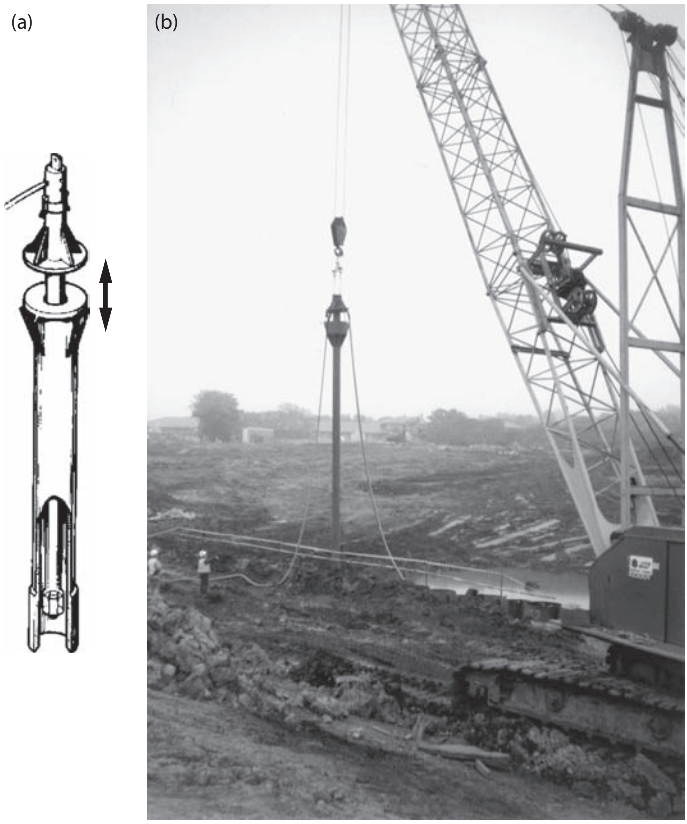


Figure 9.10 Holepuncher for installation. (a) Schematic view of holepuncher. The inner wash pipe is raised or lowered during jetting, to allow the weighted head to be used as a drop hammer. (From Somerville, S. H., Control of groundwater for temporary works. Construction Industry Research and Information Association, *CIRIA Report 113*, London, 1986: Reproduced by kind permission of CIRIA: www.ciria.org.) (b) Installation in progress. The holepuncher is suspended from a crane. (Courtesy of WJ Groundwater Limited, Bushey, U.K.)

9.4.4 Installation by rotary jet drilling

The rotary jet drilling technique (Figure 9.11) is a modern adaptation of jetting by placing tube. An excavator-mounted drill mast with hydraulic rotary head and swivel allows the placing tube to be rotated as it is jetted in. The rotary head can also be used to apply downward force to the placing tube to aid penetration. This system has been used in a range of conditions including clays, sands, sandy gravels, and weak rock such as weathered sandstones. It can be used as an alternative to the holepuncher, to avoid some of the safety hazards associated with the latter method.

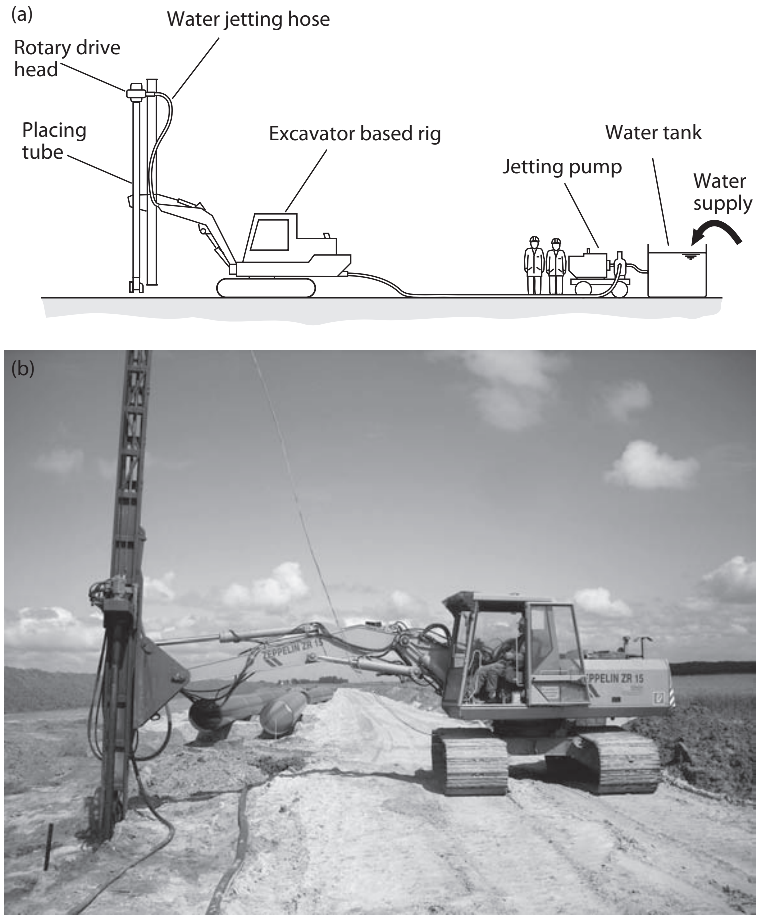


Figure 9.11 Rotary jet drilling rig. (a) Schematic view of rig. (From Preene, M. et al., Groundwater control-design and practice. Construction Industry Reasearch and Information Association, *CIRIA Report C515*. Reproduced by kind permission of CIRIA: www.ciria.org.) (b) Installation in progress. (Courtesy of Hölscher Wasserbau GmbH, Haren, Germany.)

9.4.5 Installation through clay strata

Where cohesive soils overlie water-bearing granular soils, it may be expedient to form a prebore through the cohesive soil layers using a flight auger (Figure 9.12) and then, having penetrated the clay soils, change to a water jetting technique.

Installation by jetting is likely to be difficult if clay layers have to be penetrated because, due to the cohesiveness of clays, the size of the jetted annulus outside the wellpoint and riser or placing tube is likely to be very

Figure 9.12 Hydraulic auger attachment for preboring through cohesive strata. (Courtesy of WJ Groundwater Limited, Bushey, U.K.)

restricted (or almost nonexistent) and so constrains the backwash of soil particles to the surface.

A technique used in North America to install self-jetting wellpoints in these conditions is "chain jetting." A chain is attached to the lower end of the wellpoint and wound around the riser to ground level before jetting in. The lifting and surging of the wellpoint and riser is accomplished using the chain, not the riser pipe. The chain will increase the size of the hole made through the clay and thereby create an annulus through which granular soils from below the clay layer can be washed to the surface and into which the sanding-in filter media can subsequently be installed from the base of the wellpoint to ground level.

When jetting in mixed soils (such as glacial till), the clay impediment may be further exacerbated by the presence of cobbles and boulders. If it is necessary to install wellpoints in such soil conditions, a holepuncher or rotary jet drilling rig is likely to be preferred.

Where it is difficult to install wellpoints by jetting, a site investigation borehole drilling rig may be appropriate. In such a case, boreholes may be drilled to the required depth using temporary boring casing. The wellpoint and riser are then installed centrally in the borehole and the annulus back-filled with a sand filter as the casing is withdrawn. This onerous variation on installation techniques is more likely to be appropriate to a pore water pressure reduction project but may be needed for a water lowering requirement in a highly stratified soil formation.

9.4.6 The merits of jetted hole installations

A water jetting technique using a self-jetting wellpoint, a placing tube, a holepuncher, or rotary jet drilling establishes holes without side smear and so provides a more efficient drainage hole than a hole made using a site investigation boring technique or a continuous flight auger technique. Hence, from the superiority of the resulting water abstraction properties, apart from cost considerations, the water jetting technique is to be preferred wherever practicable.

9.4.7 The need for filter media

During pumping, apart from the initial pumping period, fines should not be continuously withdrawn from the surrounding ground—so the discharge water should be clear. If this were not so, the installation is faulty for assuredly, continuous withdrawal of fines will lead to instability.

Prevention of continuous movement of fines can be achieved by using a column of filter media around each wellpoint and its riser pipe by sanding-in. This is very similar to the provision of a filter pack around the screen of a deep well (see Section 10.3) but the grading of the filter media for sanding-in of a wellpoint is not as critical as it is to a deep well installation.

The wellpoints must have sufficient flow capacity through the wellpoint screen to provide adequate water abstraction from the surrounding soil. This should not be a problem when using adequately designed wellpoints. Where there is the need to drawdown to a level close to an impermeable interface, it is prudent to use short (i.e., 0.3–0.5 m length) wellpoints to reduce the risk of drawing in air and the consequent need for repetitive adjustments to the trim valves. In a high-permeability soil, the reduced length of wellpoint will restrict its potential water-passing capacity and a compensating reduction in spacing between wellpoints will be needed. Thus, the number of wellpoints required will be greater.

The conventional coco or woven mesh stocking wrapping to the disposable wellpoints provides a filter in addition to that of the sanded-in media.

9.4.8 Sanding-in

Efficient sanding-in of wellpoints and risers helps prevent the removal of fines. The dewatering efficiency of individual wellpoints will be improved by providing preferential downward drainage paths from any overlying perched water tables which may be due to variations in vertical permeability in heterogeneous strata. Generally, a washed sharp sand, similar to a medium coarse concreting sand, will be a satisfactory filter or formation stabilizer. Sanding-in around the wellpoint and its riser pipe is essential in silty soils and fine sands but may be omitted when installing wellpoints in nonsilty coarse sands, gravelly sands, and sandy gravels. In such soils, an effective natural filter pack (see Section 10.3) can be formed by the washing action of the jetting process.

The procedure for sanding in a self-jetting wellpoint and its riser is as follows: On reaching the required depth, throttle back on the jetting pump so that the water emerging in the boil from the wellpoint hole at ground level just gently "bubbles." This upward flow of water should maintain the oversized jetted hole temporarily while sanding-in progresses. If this oversized hole is not maintained because the jetting pump has stopped, the soil may collapse around the wellpoint and the integrity of the filter column will be impaired. In general, the technique for placing the filter media is very basic, shoveling sand into the annulus which is being kept open by the "bubbling" rising water until the annulus is filled to ground level.

Sanding-in is vital to achieve an efficient installation in layered or other nonhomogeneous soils. It has been known for only two or three shovels of sand to be placed in a wellpoint annulus—this is totally inadequate!

Disposable wellpoints installed by placing tube are sanded-in in a similar manner. After the water is turned off and the top cap removed from the placing tube on reaching the required depth, the wellpoint and riser are installed centrally within the placing tube. The filter sand is placed around the wellpoint and riser within the placing tube as it is withdrawn. The level of the top of the filter sand should always be above the level of the bottom of the placing tube as it is being withdrawn. Some installation crews sand-in by adding sand around the outside of the placing tube as it is withdrawn.

The soil structure is important—is the soil mass nearly homogeneous or is it anisotropic? If the soil structure is markedly anisotropic, there will be potential for encountering a series of perched water tables or local trouble spots. Adequate sanding-in is vital to the successful drainage of anisotropic soils. Also, where a stratum of soft clay is penetrated by a self-jetting wellpoint, the clay may squeeze in before the vertical column of filter sand is placed—the filter sand may bridge at the top of the clay layer.

Table 9.1 Typical wellpoint spacings and drawdown times

Soil type	*Typical spacings (m)*	*Drawdown (days)*
Fine to coarse gravel	0.5–1.0	1–3
Clean fine to coarse sand and sandy gravel	1–2.0	2–7
Silty sands	1.5–3.0	7–21

Note: If there is a risk of residual overbleed seepage, wellpoint spacings should be at the closer end of the range.

9.5 SPACING OF WELLPOINTS AND DRAWDOWN TIMES

The theoretical number of wellpoints required for a particular project, and their associated spacings, will be indicated by the calculations outlined in Chapter 7. However, the spacing of wellpoints for simple trenching excavations is often determined from past experience of working in similar soils.

In practice, the spacing between wellpoints tends to be influenced by the spacing of the take-off points on the suction header main supplied by the manufacturer. These are mostly at 1 m centers; thus, the actual spacings will usually be at 1, 1.5, and 2 m centers. A spacing of 0.5 m can be achieved with two parallel header mains, each with 1 m take-off spacings. It can be difficult to actually install wellpoints at 0.5 m spacings in a single line; jetting in one wellpoint may blow the adjacent wellpoints out of the ground. In such cases, it can be appropriate to install the number of wellpoints equivalent to one line at 0.5 m spacings, but actually lay the wellpoints out in two parallel lines, each at 1.0 m spacing.

Although the spacing selected to achieve drawdown will depend on permeability and soil structure; program time requirements are often of primary importance. If rapid drawdown is needed, wellpoints should be installed at closer centers. Typical spacings and approximate drawdown times for a single-stage installation are given in Table 9.1.

9.6 SEALED VACUUM WELLPOINT SYSTEM

Gravity drainage to a wellpoint installation in low-permeability fine-grained soils of permeability less than approximately 1×10^{-5} to 5×10^{-5} m/s is usually very slow. The rate of drainage can be improved by sealing the annulus around the wellpoint riser at the topsoil zone, with puddled clay or a cement/bentonite mix having a putty-like consistency. This allows the suction action of the pump to generate a vacuum in the entire filter column, increasing the hydraulic gradient between the soil and the wellpoint.

This is often described as vacuum wellpointing, and is one of the methods of pore water pressure control used in fine-grained soils (see Section 5.5).

This technique can be quite effective in stratified soils, provided that proper sanding-in has been achieved from the bottom of the wellpoint to the underside of the clay plug. On occasion, it has been used effectively to reduce the moisture content of low-strength clay soils and thereby increase slope stability. The vacuum tank unit pumpset (see Section 13.2) would be the appropriate type to use for this particular duty. Ejector systems (see Section 11.2) might be considered for use in place of a vacuum wellpoint system.

The wellpoints should be closely spaced because of the limited effect of individual wells in low-permeability soils. If this technique is applied to low-permeability soils in a loose condition, the side slopes of the excavation should be protected from sudden disturbances—such as the use of any vibration techniques—as these might cause liquefaction.

9.7 WELLPOINT PUMPING EQUIPMENT

Wellpoint equipment sets are available from many specialist manufacturers. The two important aspects to be considered before ordering equipment are:

1. Long-term reliability.
2. Overall cost, including maintenance costs.

Unfortunately, some suppliers do not conform to these simple and sensible guidelines. Some supply cumbersome equipment that require cranage for handling. Other suppliers go to the other extreme, supplying flimsy equipment that is readily susceptible to damage and so has only a short useful life.

A typical medium-sized wellpoint installation will incorporate a 150-mm duty pump plus a standby pump connected to a 150-mm-sized header main (which acts as a suction manifold) connected to the trim valves of each individual wellpoint via its riser. The individual component parts have been described earlier.

9.7.1 Duty or running pumps

The total connected pump capacity must be sufficient to deal with the greater rate of abstraction required during initial drawdown. In unconfined aquifers of moderate to high permeability, the calculated steady-state pumping rate is unlikely to be adequate to achieve acceptably rapid drawdown. In such circumstances, the initial rate of pumping may be up to twice the calculated equilibrium rate of pumping required to maintain drawdown. It is common practice on start up to operate both the running and the standby pumpsets to achieve a fast drawdown.

When dealing with high flow rates or a large installation, it is prudent to provide multiple pumpsets in the system rather than a single large-output running pump because this affords greater flexibility. This often results in better fuel economy and is economical on the provision of standby pumps. Chapter 13 describes the types of pumps suitable for use with wellpoint systems.

For systems running with a single duty pump, the ideal position for the pump is at the middle of the suction header main so that water is being abstracted from an equal number of wellpoints to either side of the pump station. Where two duty pumps are used, they may be positioned adjacent to each other—thus at the end of each set's header line—and a standby pump may be positioned between them and connected so as to pump from either set (Figure 9.4). This is acceptable only if the pumping load required of either running pump is significantly below its rated capacity.

Generally, wellpoint installations are operated using diesel-driven units. However, if the running pump is to be electrically powered (perhaps to reduce noise levels), it is often acceptable and economical that the standby pump be diesel powered. Because groundwater lowering pumpsets operate 24 hours per day, fuel is a significant cost factor. Fuel consumption should be highlighted in the buildup of cost estimates. The costs of alternative sources of power (such as mains electric power) should be compared if their provision is practicable.

The fuel consumption of the double-acting piston pump (Section 13.2.1) is more modest than that of the centrifugal pump; also, the wear and tear is less because of the slower rate of movement. However, the maximum amount of lowering that can be achieved using a piston pump is generally limited to approximately 4.5 m. An efficient system operated with a vacuum-assisted self-priming centrifugal pumpset should achieve an additional 1 m or more of drawdown in similar soil conditions.

These are general indications for guidance only. Actual achievements will depend on the permeability of the soils, the mechanical efficiency of the individual pumpsets and of all the pipework connections of the installed system. Air leaks must be minimized because they can have a very significant effect on the amount of suction available at the wellpoints and thus affect the lowering that is achievable.

9.7.2 Standby pumps

In general, groundwater lowering systems should operate continuously 24 hours per day, 7 days per week. Hence, it is imperative that the installed system incorporates facilities to ensure that pumping is indeed continuous. Generally, this can be provided by connecting an additional (standby) wellpoint pump to the suction header and discharge mains with suitably placed valves for a swift changeover of operation from the running pump to the standby unit as and when required. This might be necessary in the event of an individual running pump failure or maintenance stoppage to check oil levels, etc.

A judgment should be made before commencing site work, of the likely effects of a cessation of pumping. The decision should be based on the answers to the following two questions:

1. Will a cessation of pumping cause instability of the works? If the answer to this question is “yes,” the provision of standby pumping or power facilities is essential.
2. Will a cessation of pumping create only a relatively minor mess that can be cleaned up afterward at an acceptable cost both in terms of inconvenience and delay? If the answer to this question is “yes,” and there is no safety risk, a judgment has to be made whether or not to accept the cost savings by not providing standby pumps while recognizing that a risk is being taken.

9.7.3 Operation of a wellpoint system and “trimming”

The amount of lowering of the water level that can be achieved by a wellpoint system is governed by:

1. First and foremost, the physical bounds of suction lift. Elevation above sea level and, to a lesser extent, ambient temperatures, have some input into this limitation. As ground elevation above sea level increases, available suction decreases. Also, ambient temperatures and ground elevation are of significant relevance to the rating of the engine required to drive the pump to achieve the necessary rate of pumping.
2. The hydraulic efficiency of the total wellpoint installation; this includes the pipework sizing above and below ground as well as the adequacy of the wellpoint sanding-in procedures and the air/water separation unit on the pumpset.
3. Air leaks. These can drastically reduce the amount of vacuum that is available to withdraw water from the soil via the abstraction points.

As suction is applied to self-jetting wellpoints, their ball valves are seated and the groundwater is sucked through the wellpoint screen only. The bottom of the riser pipe terminates near the bottom of the wellpoint screen so as to minimize or delay the intake of air when maximum drawdown is being achieved.

The bottom end of a disposable wellpoint is sealed and thus groundwater is sucked through the screen to the bottom of the riser pipe and, from there, to the suction header main.

As the water level at each wellpoint is drawn down to near the level of the top of the screen, there will be a risk of entraining air with water and thereby reducing the amount of available vacuum. The trim valves (Figure 9.3) at each individual wellpoint connection to the header main enable the experienced operator to adjust the amount of suction, such that the intake is predominantly water and the amount of air intake is minimized. This is necessary to ensure that the wellpoint system operates at the maximum achievable drawdown.

The adjustment of the valves on each wellpoint is known as “trimming” or “tuning” and is an important part of operating any wellpoint system. A poorly trimmed system may achieve significantly less drawdown than a system that has been trimmed correctly. Trimming is probably more of an art than a science, but can normally be mastered with experience.

As the system is trimmed (i.e., the wellpoint valves are closed or throttled), the vacuum shown on gauges on the pump and header main should increase. This is because the amount of air entering the system via the wellpoint screens is reducing, allowing the pump to generate more vacuum. When trimming a system, it can be very satisfying to see the vacuum increase as a result of your efforts. However, it must be remembered that the aim is not to maximize vacuum but to maximize drawdown and flow rate. It is important not to become obsessed with obtaining greater and greater vacuums. An overzealously trimmed system (where all the wellpoint valves are almost closed) will have a very impressive vacuum but will pump little water and generate little drawdown—this is a classic mistake made by novices when first attempting trimming!

A wellpoint that is pumping problematic volumes of air will probably be producing “slugs” or “gulps” of air and water. The momentum of each slug of water will make the wellpoint swing connector jump up and down, perhaps quite violently—this is known as “bumping.” A “bumper” is trimmed by slowly closing the valve until the flow is smooth, and then reopened slightly—the last action is vital to avoid overtrimming. A small steady flow of air along the swing connector is acceptable (provided the rest of the system is functioning well); it is probably counterproductive to attempt to trim a system so that no air at all enters the wellpoints.

Wellpoint equipment sets supplied by British, North American, and Australian suppliers usually include trim valves to regulate the rate of water/airflow to each wellpoint. However, suppliers based on the continent of Europe generally do not supply trim valves, unless specifically asked to do so. The reason for this is not clear. Southeast Asian suppliers also tend to omit trim valves and to follow continental European philosophy. The inclusion of trim valves is added cost but the authors consider that the potential for increased lowering by adjustment of trim valves, especially for a multistage wellpoint installation, is cost-effective.

9.8 WELLPOINT INSTALLATIONS FOR TRENCH EXCAVATIONS

A considerable amount of shallow-depth trenching for pipelaying is carried out worldwide. Irvine and Smith (2001) provide overall guidance to good trenching practices and include many specific recommendations concerning the support of excavations and other safety guidelines, both with and without the need for groundwater control. This book does not address the many methods of trench support. However, it is important that for each and every trench pumping installation, the basic concepts described should be observed; although depending on local conditions, some adjustments may be appropriate.

9.8.1 Single-sided wellpoint installations

The pipelaying contractor will much prefer having the header main and associated wellpoints only on one side of the trench because this will allow uninterrupted access to the plant and equipment on the other side of the spread (Figure 9.1). The basic question is how to assess whether a single-sided installation will be adequate. Water levels must be reduced to below the formation level. The critical point is the bottom of the trench on the side remote from the line of wellpoints of Figure 9.1. If the drawn down phreatic surface is below that point, a single-sided installation is adequate.

A single-sided system will be suitable only if a sufficient depth of permeable soil exists beneath the formation level and if there are no significant layers or lenses of impermeable material in the water-bearing soils, especially above the level of the base of the wellpoints.

The slope and shape of the drawdown surface is very relevant, and primarily depends on the soil permeability. The lower the permeability, the steeper the slope of the drawdown cones that individual wellpoints will establish. The spacing between wellpoints must be close enough to ensure that their individual cones of depression interact and so produce a general and fairly uniform lowering.

The dewatering contractor has no control over soil conditions on-site but does have control over the spacing between wellpoints. If the proposed excavation depth is modest and the depth of the unconfined water-bearing stratum is considerable, the wellpoints can be installed at wide spacings on long riser pipes. The use of long riser pipes has the merit of minimizing the possibility of a wellpoint sucking air and so reducing the available vacuum. This may be offset by increased difficulty in the handling of long risers during installation. Generally, this variation is more suitable for an installation that is to be pumped for a significant period than to a progressive type installation, when the pumping period will be of short duration.

Experienced engineering judgment is needed when deciding on wellpoint spacings and whether a single-sided installation will be adequate. The conditions favorable to the use of a single-sided wellpoint installation for trench excavation are:

1. A narrow trench.
2. Effectively homogeneous, isotropic permeable soil conditions that persist to an adequate depth below formation level (see Figure 9.1).
3. Trench formation level is not more than approximately 5 m below standing groundwater level. The actual depth achievable depends on the permeability of the water-bearing soil beneath the formation and the overall pumping efficiency of the installation.

For trenching in excess of approximately 5 m deep, it is possible to use a lower stage of wellpointing as described in Section 9.10, provided that there is sufficient depth of permeable soil beneath the final formation level and the width of the wayleave is sufficient. The sides of the trench excavation must be safely supported throughout.

Single-sided systems work well in isotropic soil conditions. In practice, soils are often heterogeneous and anisotropic. In such conditions, double-sided systems, in which wellpoints are installed on both sides of an excavation, may be more effective.

9.8.2 Double-sided wellpoint installations

The conditions likely to require the use of a double-sided wellpointing (Figure 9.13a) for trench excavation are:

1. A relatively wide trench.
2. Trench formation level of 4.5 to 6 m or more below standing groundwater level.
3. Impermeable stratum close to the formation level, or low-permeability layers or lenses present above the formation level.

If there is an impermeable stratum (such as a clay layer) at or close to formation level (Figure 9.13b), even if a double-sided wellpoint system is used, there may be some residual seepage or overbleed near the interface between the overlying permeable stratum and the underlying impermeable

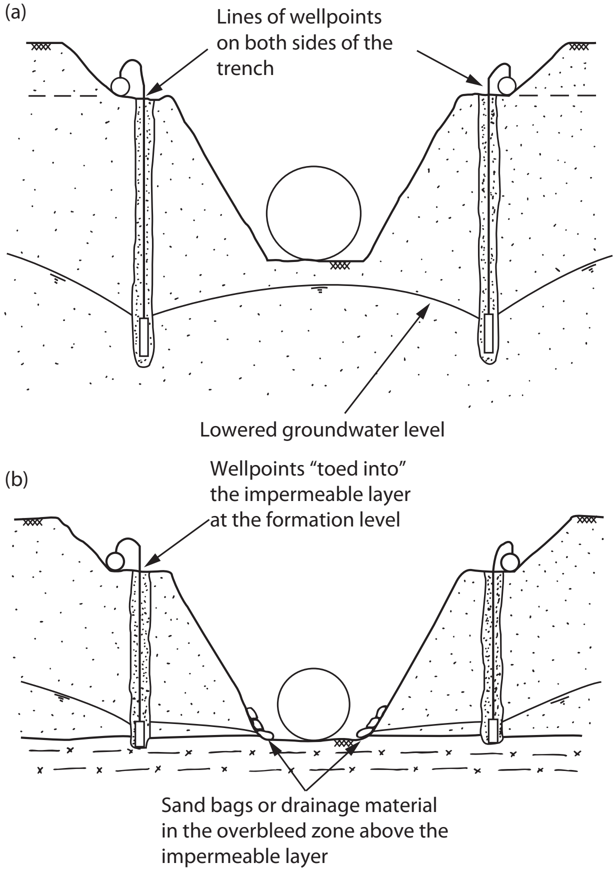


Figure 9.13 Double-sided wellpoint installation. (a) Permeable stratum extends below the formation level. (b) Impermeable stratum present near the formation level.

soil. In this situation, particular thought must be given to the wellpoint depth and spacing. The wellpoints must be "toed into" the impermeable stratum—in effect, they are installed to penetrate a few hundred millimeters into the clay, to form a sump of sorts around each wellpoint to maximize drawdown. This requires careful installation—overzealous "toeing in" will result in the wellpoint screen being installed too deep into the clay, thereby becoming clogged and ineffective. The wellpoints should be installed at rather closer spacings than normal to try and intercept as much of the overbleed seepage as possible.

Even if these measures are adopted, some overbleed seepage is likely to pass between the wellpoints and enter the trench. It is essential to control the overbleed so that pore water pressures do not build up and that

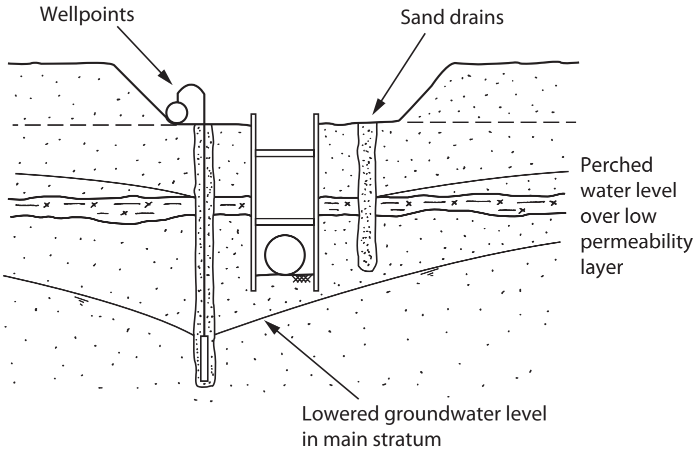


Figure 9.14 Single-sided wellpoint installation with sand drains to aid control of perched water table.

the water flow does not continuously transport fines and risk instability of the sides of the trench. Some form of sand-filled permeable bags, granular drainage blanket, or geotextile mesh should be placed as indicated in Figure 9.13b. This will allow the water to flow into the excavation without the build-up of pore water pressures and movement of fines. Thus, the stability of the trench will be preserved. The water flowing to the trench must be continuously removed by conventional sump pumping to prevent standing water from building up in the working area.

There is a hybrid of single and double-sided systems that can be used when trenching in extensive permeable strata where an impermeable layer exists above the formation level (see Figure 9.14). These conditions could be dealt with by using a double-sided wellpoint installation but this could be an encumbrance to the contractor's excavation and pipelaying activities. The alternative is to have a single-sided wellpoint installation on one side of the trench excavation with vertical sand drains on the other side. The sand drains consist of a series of holes jetted at diameters and spacings similar to the wellpoints and penetrating to below the low-permeability layer. These are backfilled with sand of the same grading as that used for the sanding-in of the wellpoints. These will provide downward drainage of the perched water on the other side of the trench. This should reduce troublesome overbleed on the side of the trench remote from the wellpoints.

9.8.3 Progressive installation for trench works

The same principles applicable to wellpoint systems for static excavations are pertinent to rolling trench excavations. There are additional

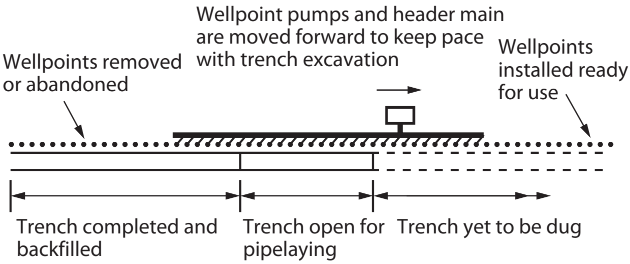


Figure 9.15 Method of wellpoint system progression for trench works.

complications because of the need to progress the groundwater lowering system to keep pace with the trench works. The initial lowering syndrome to establish drawdown is always at the head of the progressively advancing installation.

Generally, the equipment length for a wellpoint system in a rolling trench excavation should be about three to four times as long as the planned weekly advancement of the trench (Figure 9.15). This allows time for the extraction of wellpoints, risers, and manifolds behind completed work, their progressive installation ahead of the work, and—for the least certain factor—the time to establish drawdown of the groundwater ahead of trenching excavation and subsequent pipelaying and backfilling.

Where the soil is of low permeability, the time for drawdown will be protracted, thus in such soils, it is prudent to allow for a greater length of operating equipment to be installed and operating ahead of the length of an open excavation.

9.9 WELLPOINTING FOR WIDE EXCAVATIONS

Where a wide excavation is required, a perimeter wellpoint installation alone may not be sufficient to establish adequate lowering at the middle of the excavation. The lower the permeability, the more likely it is that the amount of lowering at the center of the excavation will be insufficient. It might be prudent, even necessary, to install additional wellpoints within the excavation area.

9.10 WELLPOINTING FOR DEEPER EXCAVATIONS

9.10.1 Long risers and lowered header mains

If the excavation width is modest, but the required drawdown is greater than achievable by a single stage, and the thickness of permeable soils beneath the formation level is significant, drawdown can be increased by installing wellpoints on longer than normal 6 m risers to an adequate depth below the formation level.

The additional drawdown is achieved by initially pumping the wellpoints from pumps and header mains laid at about standing groundwater level. Pumping of this initial installation will establish some lowering of the water level. An excavation is then made down to the lowered water level and a second suction header main is installed at the lower level. This lower main is pumped while the upper suction main is still active. Progressively, each individual wellpoint (having shortened its riser length) is disconnected from the upper header main and connected to the lower header main. Pumping should be continued from the original upper main until all wellpoint risers have been shortened and connected into the lower main. Using the standby pump from the upper header main to begin pumping on the lower header main can reduce the number of pumps required for this method. When all the wellpoints have been connected to the lower main, the upper pump can be moved down to act as a standby unit for the lower main. The upper header main is then dismantled and removed.

This process could be described as a two-stage installation having only one stage of installation of the wellpoints and risers (see Section 9.11 for description of a case history where this procedure was used).

9.10.2 Multistage wellpointing

Multistage wellpoint installations (Figure 9.6) can be used for deep excavations as an alternative to deep wells (see Chapter 10) or ejectors (see Section 11.2). If a multistage wellpoint system is used, it is necessary to make sufficient allowance for side slopes or batters to excavate safely to formation depth and berms to support header mains of lower stages.

The recommended site procedure for the installation of multistage sets is as follows:

1. Excavate to about the standing water level; it may be possible to excavate to approximately 0.5 m below the water level.
2. Install and connect the first stage wellpoint system around the perimeter, making due allowance for subsequent slope batters and berms.
3. Pump continuously on the first stage system.
4. Excavate to approximately 0.5 m above the lowered water level.
5. Install and connect the second stage wellpoint system around the perimeter that likewise allows for excavation batters and berms for subsequent stages.
6. Pump continuously on the second stage system as well as continuing to pump on the first stage.

7. Again excavate to approximately 0.5 m above the level to which water has been lowered by pumping on both first and second stages.
8. Continue the sequence of excavation, wellpoint installation, and pumping until the formation level is reached.

This sequence of operations will entail short halts in excavation as each further intermediate stage level for wellpoints and header main, etc., is installed for further lowering of the groundwater.

At each stage, in addition to the installation of the wellpoint system, a number of observation wells should be installed to monitor the lowering being achieved and to indicate the depth to which the next stage that excavation can be taken.

Often, when pumping on multistage systems of more than two stages, the first stage pumping output declines rapidly as the third stage is brought into operation. When this happens, the top stage pumps can be stopped and transferred down to the fourth stage if there is one: or can be connected to the third stage as standby units. The wellpoints and risers of the top stage

Figure 9.16 Four-stage wellpoint system. The sand being dewatered is slightly cemented, which allows the side slopes to stand very steeply in the short term. The cementing is sufficiently weak that the ground unravels in the event of water inflow but in a "dewatered" state can stand almost vertically. (Courtesy of WJ Groundwater Limited, Bushey, U.K.)

should not be extracted because it may be necessary to reactivate the upper stage(s) as the structure is being built up and backfill is placed. The recovery of the groundwater level must be controlled so that at every stage of the building of the works, there is no risk of flotation of the partially completed structure due to an uncontrolled rise of the groundwater.

Three- and four-stage wellpoint installations have been operated satisfactorily for deep excavations (Figure 9.16). There have been, and will be in the future, many projects in which a multistage wellpoint system is an economic solution to a particular groundwater lowering problem. Consideration should also be given to the fact that the degree of expertise required for the installation and operation of a wellpoint system is less sophisticated than that appropriate for a deep well or ejector system. Hence, there may be occasions or geographical locations where the use of a wellpoint system will be the more appropriate choice for a deep construction project.

9.11 CASE HISTORY: DERWENT OUTLET CHANNEL, NORTHUMBERLAND

In the early 1960s, the Sunderland & South Shields Water Co. awarded to John Mowlem & Co. Ltd. a contract for the construction of an earth dam about fifteen miles to the southwest of Newcastle on Tyne, in the valley of the river Derwent (see Rowe 1968; Buchanan 1970; Cashman 1971). This required the formation of a spillway outlet channel about 240 m long and approximately 8 m deep on average. Laminated glacial lake silts of low permeability were revealed at the upstream end (Figure 9.17).

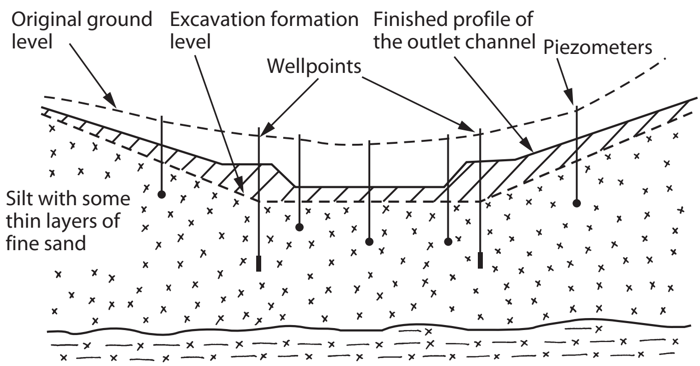


Figure 9.17 Typical cross-section of the Derwent outlet channel. (Courtesy of Northumbrian Water Limited, Durham, U.K.)

Grading curves for soil samples obtained from site investigation boreholes confirmed the existence of difficult to stabilize soils (Figure 9.18). In addition, the varved soil structure was complicated.

Initially, an excavation was made to modest depth below the piezometric surface in the glacial lake deposits. This aided the assessment of excavation problems likely to be encountered and to determine appropriate methods for achieving a stable excavation for the construction of the outlet channel. The conditions exposed were not encouraging (Figure 9.19). There were many washouts in the excavation faces and outwash fans due to seepages that continuously transported fines. Also, a series of piezometer tubes installed in the excavation had to be extended to some height above the excavated level because the pore water pressure at depth was artesian relative to the excavation floor. The excavation surface was so unstable that duckboards were needed to be able to take the measurements in these piezometers.

Careful undisturbed sampling of these glacial lake soils revealed, within the layered soil structure of these deposits, some very thin layers of more permeable material that could act as preferential drainage layers to influence the reduction of pore water pressures of the total soil mass. It was agreed by the client, his specialist advisors, and the contractor that a trial wellpoint installation, with careful attention to sanding-in, should be undertaken.

Because the amount of lowering of the groundwater required to reach formation level was greater than that which could be achieved with a single-stage wellpoint installation, the wellpoints were installed at 1.8 m

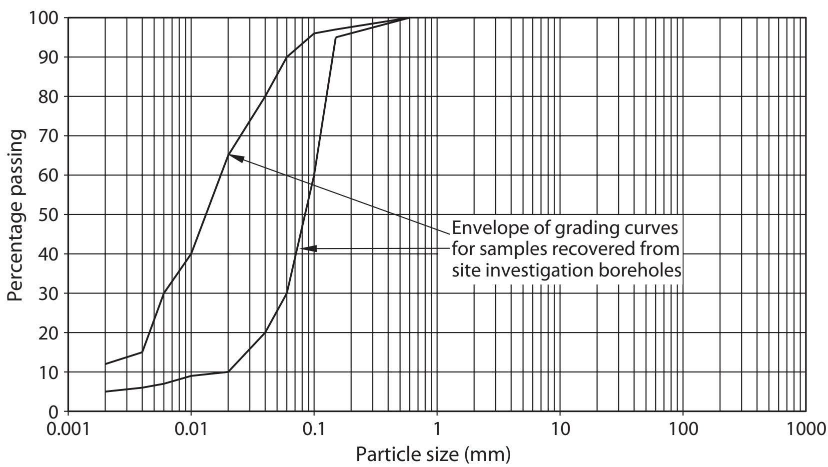


Figure 9.18 Envelope of gradings of glacial lake soils, the Derwent outlet channel. (Courtesy of Northumbrian Water Limited, Durham, U.K.)

Figure 9.19 Initial excavation (showing unstable conditions) for Derwent outlet channel. (Courtesy of Northumbrian Water Limited, Durham, U.K.)

centers on extra length riser pipes (see Section 9.10). Pumping from this initial installation achieved approximately 3 m of lowering of the water level within a period of about a week. This created a surface of sufficient firmness to support an excavator and 2.75-m-deep trenches were then opened beside the two lines of wellpoints. A duplicate suction header main was laid at this lower level and progressively each riser length was shortened and connected into the lower active pumping main. By this means lowering to formation level for the outlet channel was achieved. The vertical column of sanding-in material around the wellpoints provided the all-important downward drainage for multilayer perched water tables.

The lower header main is visible in the foreground of Figure 9.20. As excavation to formation level moved forward, this main was progressively supported on scaffolding. The channel was shaped to the required formation and side slopes (see Figure 9.17), and as shaping proceeded, a sand blanket was quickly laid to permit the drainage of pore water but at the same time preventing the removal of fines.

The principles observed were:

- Shape the excavation to just below the formation level and form the required side slopes.

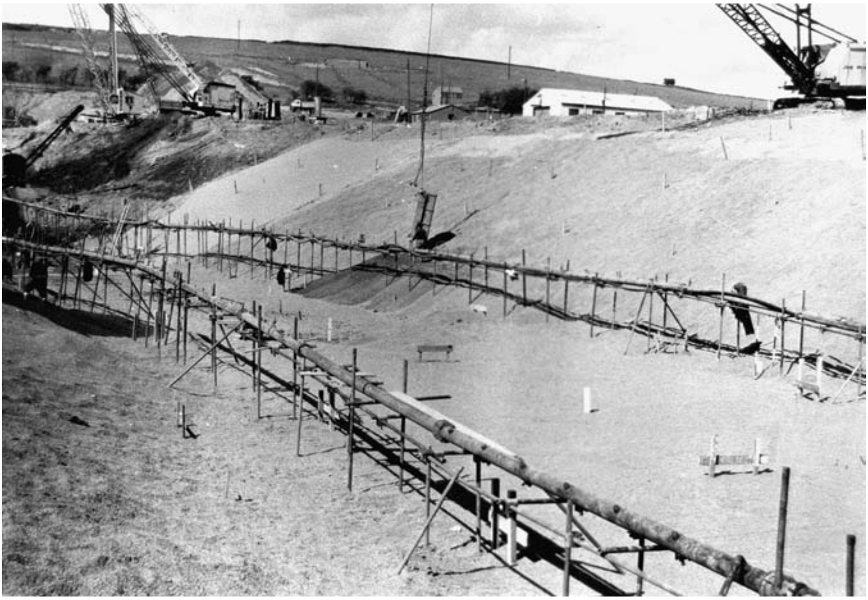


Figure 9.20 Workable conditions, following wellpoint pumping, under which the channel was actually formed. (Courtesy of Northumbrian Water Limited, Durham, U.K.)

- As the side slopes are formed, immediately and progressively blanket them to allow relief of water seepages, thus preventing any buildup of pore pressures.
- Do not allow continuous transportation of fines, this can only lead to slope instability.
- Remove all seepage waters by controlled (i.e., filtered) sump pumping.

The message is—study and understand the soil structure and observe the basic installation guidelines. Thereby the almost impossible can become quite possible.

Chapter 10

Deep well systems

10.1 INTRODUCTION

A deep well system consists of an array of bored wells pumped by submersible pumps. The wells act in concert: the interaction between the cone of drawdown created by each well results in groundwater lowering over a wide area. Because the technique does not operate on a suction principle, greater drawdowns can be achieved than with a single-stage wellpoint system. This chapter addresses the temporary works deep well groundwater lowering system and describes good practices to be used during installation and operation.

The principles of the method and the stages in well design are discussed. The methods used for well drilling, installation, development, and operation are outlined, and some practical problems with the operation of deep well systems are presented. The vacuum deep well and bored shallow well systems, which are variants of the deep well method, are described briefly. The chapter ends with a case history of a large-scale temporary works deep well system.

10.2 DEEP WELL INSTALLATIONS

A deep well system consists of a number of wells pumped by submersible pumps. Each well consists of a bored hole (typically formed by a drilling rig) into which a special well liner is inserted. The liner consists of plastic or steel pipe, of which a section is slotted or perforated to form a well screen to allow water to enter; other sections consist of unperforated pipe (the well casing). Generally, deep well systems are installed in drift deposits, and the annulus between the borehole and the well screen/casing is backfilled with filter media or formation stabilizer to form what is known as the filter pack.

The wells are generally sited just outside the area of proposed excavation (although for very large excavations, wells may be required within the main excavation area as well as around the perimeter). A deep well system has individual pumps positioned near the bottom of each well; usually, the pumps are borehole electro-submersibles (see Section 13.5). The well screen and casing provides a vertical hole into which a submersible pump attached to its riser pipes can be installed (and recovered, as and when required).

A typical deep well system (Figure 10.1) consists of several wells acting in concert. Each well creates a cone of depression or drawdown around itself, which in a high-permeability aquifer may extend for several hundred meters. The interaction between the cones of drawdown from each well produces the drawdown required for excavation over a wide area. Apart from pumping tests, deep wells are rarely used in isolation or individually; the method relies on the interaction of drawdowns between multiple wells.

The components making up an individual temporary works well are shown in Figure 10.2. Generally, for most temporary works requirements:

1. The well screen and casing sizes will be in the range of 150–300 mm diameter. The well screen and casing are typically plastic, with steel being used only rarely.
2. The drilled borehole sizes will be in the range of 250–450 mm diameter.

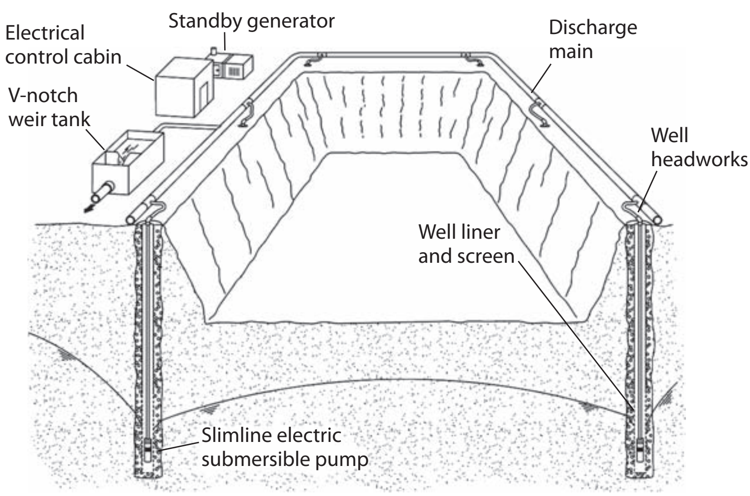


Figure 10.1 Deep well system components. (From Preene, M. et al., Groundwater control—design and practice. Construction Industry Research and Information Association, *CIRIA Report C515*, London, 2000. Reproduced by kind permission of CIRIA: www.ciria.org.)

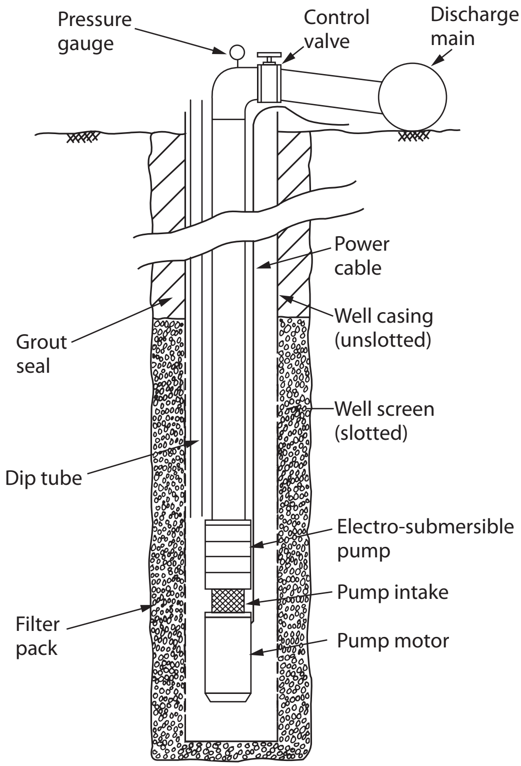


Figure 10.2 Schematic section through a deep well.

3. The well depths will be in the range of 10–50 m. Occasionally, wells are drilled to greater depths, especially for shaft or tunnel construction projects.
4. The soils through which the wells are bored are usually drift deposit soils. Occasionally, weak rock formations are encountered (such as a weak sandstone). The screened length in a rock formation may not need a well screen and filter pack; thus, the required size of bore may be smaller.

The vital feature of the deep well system compared to the single-stage wellpoint is that the theoretical drawdown that can be achieved is limited only by the depth of the well and soil stratification. The wellpoint method (see Chapter 9) is limited by the physical bounds of suction lift. In contrast, the drawdown of a deep well installation is constrained only by the depth/level of the intake of the pump(s)—provided, of course, that the power of the pump is adequate to cope with the total head from all causes. Hence, the rated output of the installed pumps should match the anticipated well yield.

The energy costs of operating a deep well installation are likely to be competitive due to the greater efficiency of borehole pumps compared with the total system efficiency of a multistaged wellpoint installation.

The well screens, pumps, and other materials are similar to those used for water supply wells. However, because the working life of a temporary works well will almost always be significantly less than the life of a water supply well, temporary works wells can be constructed by rather cheaper and simpler methods. Moreover, the onerous health regulations to control the risk of water contamination during construction of water supply well installations are mostly inappropriate to temporary works wells.

The initial cost of installing a deep well system is significant. A high standard of expertise in the design and control of installation procedures is required to ensure that the appropriate good practices are implemented throughout and to promote optimum and economic performance.

10.3 DESIGN OF WELLS FOR GROUNDWATER LOWERING

There are three major factors to be considered when designing an individual temporary works well:

1. The depth of the well and screen length
2. The diameter of the borehole and well screen
3. The filter media to be used

The design of a well will generally be done after the design of the overall groundwater lowering system has estimated the total flow that must be pumped to obtain the required drawdown for the excavation (see Section 7.7). The specification of well yield and spacing is outlined in Section 7.8. The following sections describe some of the practical issues associated with the design of individual deep wells.

10.3.1 Depth of well

In essence, the well must be deep enough to

1. Yield sufficient water so that all the wells in concert can achieve the required flow rate and, hence, the required drawdown.
2. Be of sufficient depth to penetrate the geological strata in which groundwater pressures are to be lowered (this is especially important if deep, confined aquifers are present).

The design of a well that ensures adequate yield is described in Section 7.8. The depth requirements are generally met by a consideration of the soil stratification at the site. There are certain rules of thumb about well depth, built up over many years of practice, and well designs should be compared with these to check for gross errors. If the aquifer extends for some depth below the base of the excavation being dewatered, the wells should be between 1.5 and 2 times the depth of the excavation. Wells significantly shallower than this are unlikely to be effective unless they are very closely spaced (analogous to a wellpoint system).

If the geology does not consist of one aquifer that extends to a great depth, this may affect well depth. If the aquifer is relatively thin, screen lengths will be limited and a greater number of wells will be required. If there is a deeper permeable stratum that if pumped could act to "under-drain" the soils above (see Section 7.6), it may be worth deepening the wells (beyond the minimum required) in order to intercept the deep layer.

It is always prudent to allow for a few extra wells in the system over and above the theoretically calculated number. This not only allows for some margin for error in design but also means that the system will achieve the desired drawdown if one or two wells are nonoperational due to maintenance or pump failure.

10.3.2 Diameter of the well

In general, the diameter of the well will be chosen to ensure that the borehole electro-submersible pump (see Section 13.5) to be used will fit inside the well screen and casing and that any necessary filter media can be placed around the well screen. This will allow the necessary drilled diameter of borehole to be determined. Sometimes, when working in remote locations or in developing countries, the selection has to be made in reverse, beginning with the borehole diameter that can be drilled by locally available equipment and then working backwards to the size of pump that can be accommodated.

The starting point for determining the size of bore is the diameter of the pump to be installed in the well screen. Having determined the expected individual well yield (see Section 7.8) at the steady-state rate of pumping, a pump should be selected from a manufacturer's or hirer's catalog that has a maximum rated performance of between approximately 110% and 150% of the steady-state flow rate at the anticipated working head. This allows for some additional pumping capacity to help establish the drawdown.

The pump manufacturer's catalog will list the minimum internal diameter of well screen necessary to accommodate the pump to be used, assuming the wells are perfectly straight and plumb. In practice, most wells deviate from the ideal alignment, and using a slightly larger screen diameter reduces the risk of a pump getting stuck down a well. Some general guidance on well screen diameters is given in Table 10.1. The recommended

Table 10.1 Recommended well screen and casing diameters

Maximum submersible pump discharge rate (L/s)	*Recommended minimum internal diameter of well screen and casing[a] (mm)*	*Recommended minimum diameter of boring[b] (mm)*
5	125–152	250–275
10	152–203	300–325
15	165–250	300–375
20	180–250	300–375
25	203–300	325–425
44	250–350	375–475

[a] The diameter will depend on the external dimensions of the pump used.
[b] The minimum diameter of boring is based on a nominal filter pack thickness of 50 mm. Slightly smaller diameters may be feasible if a natural filter pack can be developed in the aquifer.

minimum well screen diameters are generally larger than those quoted by the pump manufacturers. Even so, if a well has a large amount of deviation, even a very small pump may become jammed at the tight points in the well.

Knowing the required minimum internal diameter, select an appropriate well casing and screen from the manufacturer's catalogs. The standard sizes of well screens are unlikely to match exactly the internal diameter needed for the pump; thus, the next available size up of well screen and casing is used.

Typically, the wells used for groundwater lowering will have a "filter pack" of sand or gravel placed in the annulus between the borehole wall and the well screen. Preferably, the annular thickness of the filter pack should be approximately 75 mm (but never less than approximately 50 mm or more than 100–150 mm). Thus, the minimum size of the borehole to form the well should be the external diameter of the well screen and casing (including at joints where the diameter may be greatest) plus twice the thickness of the filter pack. This diameter is unlikely to exactly match the standard sizes of drilling equipment; thus, the borehole should typically be drilled using the next size up of drill bits, taking care to check that this does not result in filter pack thicknesses in excess of 150 mm.

10.3.3 Types of well screen

In most cases, the well screens used on dewatering projects will be proprietary products manufactured specifically for the purpose. Around the world, there are many companies that specialize in the production of well screens and casings in various diameters, materials, and specifications and can provide materials for dewatering projects. The well casing (the unperforated sections) and well screen (perforated sections) are typically provided in sections of between 1- and 6-m length. During installation into the well, the sections will be joined together by either threaded joints or glued joints (for plastic screens) or welding (for steel screens). Various fittings are available for casing and screens, including base caps, lifting heads, and adaptors.

One of the most common types of well screen used on dewatering projects is plastic well screens formed from either unplasticized polyvinyl chloride (uPVC) or high-density polyethylene (HDPE). The base pipe is perforated with regular arrays of linear slots of a specified size (Figure 10.3a). The most common commercially available slots sizes are 0.5–3 mm.

Continuous slot well screens have a single continuous slot (of constant width) running in a spiral round the pipe diameter. The most well-known configuration is steel screens, where the slot is formed by a single triangular or trapezoidal-shaped wire welded in a spiral around a base pipe or cylindrical structure of rods. The slot is formed between each loop of wire (Figure 10.3b). These steel screens are typically very strong and are hydraulically very efficient due to the long slot length on each length of screen. The disadvantage of these steel screens is their relatively high cost compared to conventional plastic well screens. Continuous slot screens are available in stainless steel, galvanized, and carbon steel, as well as in plastic.

Well screens are also available with prefitted or preformed filter packs. These are fitted with gravel filter media of specified particle size, avoiding the need for a conventional filter pack. This can be useful in situations where there may be practical difficulties in installing conventional filter packs. The two main types of prefitted filter packs are

1. *Resin-bonded screens*. Specific granular filter media are bonded directly to the slotted well screen in a factory-based process (Figure 10.3c).
2. *Prepacked screens*. The gravel filter medium is held in place (packed) between two specially matched concentric screens.

In some cases, preformed filters are formed by wrapping geotextile mesh around a slotted base pipe. The most hydraulically efficient designs provide a high open area by placing a very coarse spacer mesh under the filter mesh itself to allow water to flow around the outside of the base pipe beneath the filter mesh. Guidance for selection of the size of openings in the filter mesh can be obtained from the manufacturers. Once selected, the mesh is normally applied to the screen in the factory. Granular filters or formation stabilizers may be used in conjunction with geotextile screens.

When selecting a well screen and casing, care must be taken to ensure that the wall thickness of the base pipe forming the screen and casing is adequate to withstand collapse pressures from soil and groundwater loadings. There have been occasions when thin-walled plastic well screens have collapsed during development and pumping. In most of these cases, further problems were avoided by installing additional wells with thicker-walled

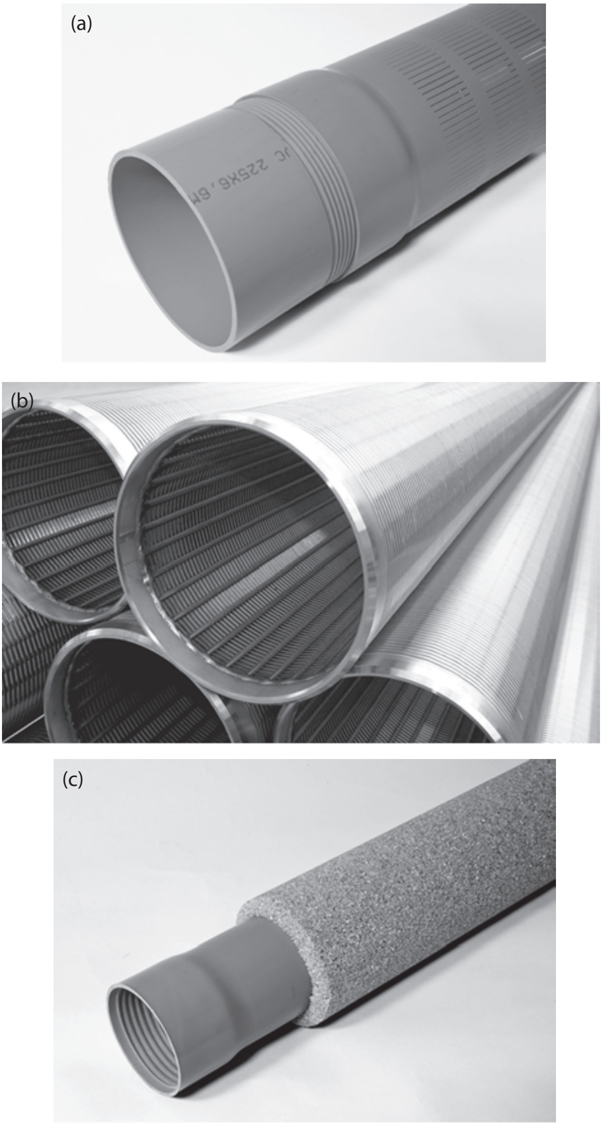


Figure 10.3 Types of well screen. (a) Perforated plastic screen. (Courtesy of Boode Water Well Systems, Zevenhuizen, The Netherlands.) The screen is fitted with threads for jointing. (b) Stainless steel continuous slot screen. (Courtesy of Johnson Screens, Availles-en-Châtellerault, France.) The screen is fitted with weld rings for jointing. (c) Perforated plastic screen with preformed filter pack. (Courtesy of Boode Water Well Systems, Zevenhuizen, The Netherlands.) This is an example of a preformed filter pack, with the filter media bonded in place with resin. The screen is fitted with threads for jointing.

screens—the screen manufacturer should be able to provide guidance. Collapse of steel well screens is rare, but steel well screens are not commonly used for temporary works wells on cost grounds.

10.3.4 Design of filter media and slot size

A temporary works groundwater lowering well (Figure 10.2) typically consists of a well screen and casing installed centrally inside a borehole formed by a drilling rig. The annulus between the screen and the borehole wall is filled with granular filter media to form a "filter pack."

The filter media must be selected (based on the particle size distribution of the aquifer material) to meet the following two conditions:

1. To be sufficiently coarse so that the filter pack is significantly more permeable than the aquifer to allow water to enter the well freely
2. To be sufficiently fine so that the finer particles are not continually withdrawn from the aquifer

The selection of any filter media has to be a compromise between these two conflicting conditions. Concentrating on condition 1 will give a high yield well, but with an increased risk of continuously pulling sand or fines into the well. Concentrating on condition 2 will prevent movement of fine particles but may restrict well yield.

An additional requirement is that the material chosen must be suitable for placement in wells (see Section 10.6) with minimum segregation; filter media of uniform grading are preferred for this reason.

There have been numerous theoretical and practical studies of design methods for granular filters in relation to water supply wells and for dams. Construction Industry Research and Information Association (CIRIA) Report C515 (Preene et al. 2000) summarized suitable criteria for the design of granular filters for dewatering purposes as follows:

1. $D_{15\text{filter}} > 4 \times D_{15\text{aquifer}}$. This satisfies condition 1. For widely graded materials, this should be applied to the finer side of the filter grading envelope and the coarser side of the aquifer grading envelope. In addition, the filter material should contain no more than 5% of particles finer than 63 μm.
2. $D_{15\text{filter}} \leq 5 \times D_{85\text{aquifer}}$. This satisfies condition 2 and is known as Terzaghi's filter criterion. For widely graded materials, this should be applied to the coarser side of the filter grading envelope and the finer side of the aquifer grading envelope.
3. $U_{\text{filter}} < 3$. This allows the filter to be placed without risk of segregation. U is the uniformity coefficient ($U = D_{60}/D_{10}$); very uniform materials (consisting of only a small range of grain sizes) have a low U. If

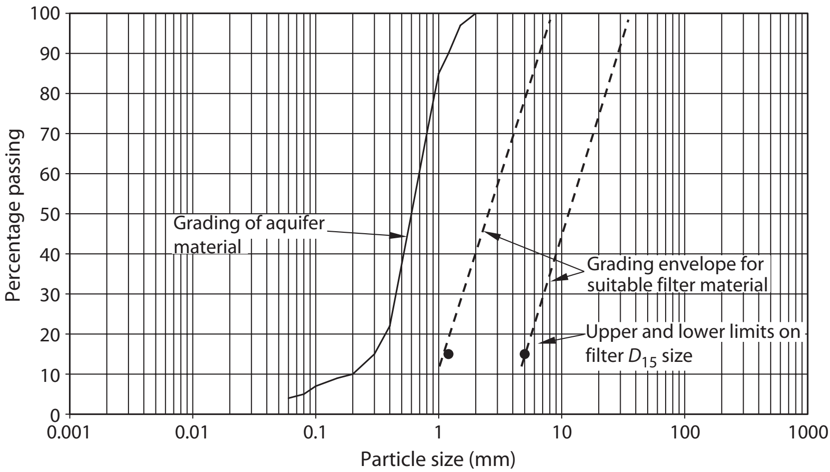


Figure 10.4 Aquifer and filter grading envelopes.

U is greater than 3, there is a risk of segregation during placement, and the filter material should be placed carefully by a tremie pipe (see Section 10.6).

The application of these criteria to an aquifer grading would produce a filter grading envelope, as shown in Figure 10.4. A filter material that falls within the envelope is then selected from those available.

The application of these criteria to real aquifers results in a relatively narrow range of materials that are suitable for use as granular filter media. At one end of the range, a relatively low-permeability silty sand aquifer might need a 0.5–1.0 mm filter sand, while at the other end, a very high permeability coarse gravel might need a 10–20 mm pea gravel. Materials outside these ranges are rarely used. Rounded uncrushed aggregates are generally considered to have higher permeabilities than the same grading of angular crushed material. For this reason, rounded materials are preferred for use as filter media.

The slot size (the minimum width of the slot) of the well screen should be chosen to match the filter and avoid large percentages of the filter material being able to pass into the well. In general, the slot size should be approximately equal to $D_{10\text{filter}}$. There is another condition that must be considered when specifying the slot size on the well screen—there must be enough total area of slots (known as the "open area") to allow the desired flow to pass through the screen. Open area is defined as the total area of slots or apertures expressed as a percentage of the total area of well screen. In general terms, the open area is much greater for well screens with larger slot sizes than smaller ones. The slotted well screens commonly used for temporary works wells typically have open areas of between 5% and 20% for fine and coarse slots, respectively. The manufacturer's catalogs should indicate the actual percentage of open area for the slot size and screen type in use.

Sterrett (2008) recommends that sufficient open area is provided to ensure that the average screen entrance velocity (well discharge divided by total area of screen apertures) is less than 0.03 m/s. Parsons (1994) has argued that this "entrance velocity" approach has little theoretical basis. Nevertheless, it is an established rule of thumb that produces wells that perform at least adequately in many situations.

When the filter is placed, it may be relatively loose and is likely to compact a little during development. Accordingly, the filter pack when placed must extend to some height (normally at least 0.5–1 m) above the top of the slotted well screen to ensure that the aquifer material does not come into direct contact with the well screen if the filter pack compacts. Sometimes, permanent tremie tubes are left in place to allow the filter pack to be topped up by the addition of more material.

There are some types of granular soils when it is not necessary to introduce artificial filter media, but where it is possible to directly develop the aquifer, remove the finer particles and form a natural filter pack in the aquifer immediately outside the well screen. This method can be employed in coarse well-graded soils such as sandy gravels. It can give cost savings by allowing a well borehole to be drilled at a smaller diameter for a given well screen size—the space for a filter pack is not needed, and the aquifer material is allowed to collapse directly onto the well screen. According to Clark (1988), soils may be appropriate for natural filters if $D_{40\text{aquifer}} > 0.5$ mm and $U_{\text{aquifer}} > 3$. Natural filters are not appropriate for uniform fine-grained soils, where there are no sufficient coarse particles in the soil to form an effective filter structure. The slot size must be chosen carefully when proposing to use natural filters. CIRIA Report C515 (Preene et al. 2000) suggests that a slot size of $D_{40\text{aquifer}}$ to $D_{50\text{aquifer}}$ is acceptable in most cases, but if the maximum yield is required from very widely graded soils, a slot size in the range of $D_{60\text{aquifer}}$ to $D_{70\text{aquifer}}$ might be considered.

If wells are installed in weak rocks (e.g., weathered chalk), where flow is predominantly from fissures, a filter pack is not needed to prevent movement of fine particles. However, it is good practice to fill the annulus between the well screen and borehole wall with a coarse permeable filter gravel that acts as a formation stabilizer. The formation stabilizer prevents weaker blocks of aquifer rock from collapsing against and distorting the well screen. Formation stabilizers should be highly permeable to allow free passage of water; 10–20 mm pea gravel is often used.

In addition to conventional granular filter media placed during well installation, preformed filters (fitted at manufacture) are also available (see Section 10.3.3).

As discussed earlier, well filters are designed against two conflicting criteria: the need to be highly permeable and the need to prevent continuous movement of fines. Add to this the problems that aquifers are variable and heterogeneous (a filter that works on one well may not work on one at the other end of the site) and that suitable filter materials may not be available locally, and it is obvious that experienced judgment is needed for filter design. The problem of aquifer variability can be addressed by carrying out a thorough site investigation. The problem of limited availability of suitable filter material (especially on remote sites) may require less than ideal filters to be used. In such cases, a series of trial wells using local materials may be appropriate. If the locally available material does not give acceptable well performance, the use of preformed (e.g., geotextile or resin-bonded) filters might be considered, because shipping costs for these materials may be less than for bulky filter gravels.

10.4 CONSTRUCTING DEEP WELLS

The methods used to construct wells for temporary works groundwater lowering purposes have much in common with those used to form water supply wells, but there are some differences in techniques and equipment. A general appreciation of well drilling methods can be gained from some of the publications in water supply (Stow 1962, 1963; Cruse 1986; Rowles 1995).

There are four main stages in forming a groundwater lowering well:

1. Drilling of a borehole
2. Installation of materials (screen, casing, and gravel)
3. Development of the well
4. Installation and operation of the pump

These are described in the following sections.

10.5 DRILLING OF WELL BOREHOLES

Many methods are available for the formation of well boreholes. The technique selected will depend on the type of equipment available in the territory, the expertise of the well boring organization selected, and the soil and groundwater conditions anticipated.

Three main methods are commonly used to form well boreholes:

1. Cable tool percussion drilling and variations on this technique, generally requiring the use of temporary boring casing to support the sides of the borehole.
2. Wash boring or water jet drilling, sometimes with the addition of compressed air. This method is very similar to the installation of wellpoints by jetting using a holepuncher.

3. Rotary drilling, either by direct or reverse circulation. The drilling fluid may be water or air with specialist additives that help support the sides of the borehole—this is typically employed when drilling drift deposits.

For temporary works wells, cable tool percussion drilling is limited to well depths of approximately 20–35 m but has been used down to a 50-m depth on occasion; for water supply wells, the method has been used to considerably greater depths. Wash boring and holepuncher equipment have been used to install wells to approximately 35-m depths. Rotary drilling rigs can cope with significantly greater depths if necessary.

The wash boring and rotary drilling techniques, when used appropriately, generally provide more productive wells and require less development. The effectiveness (i.e., potential yield) of a rotary drilled hole depends greatly on the properties of the drilling fluid used and the adequacy of the removal of the drilling residues on completion of the hole—this often gives rise to much debate between groundwater lowering specialists, drilling contractors, and drilling additive suppliers about the relative merits of the various marketed drilling slurries. The merits of various types of drilling fluids and additives are discussed by Sterrett (2008). Adequate development of the well on completion is vital to ensure its best performance (see Section 10.7).

Throughout the well drilling operation, the arisings from the drill hole should be observed and logged to determine if soil conditions are as expected. If not, it may be prudent to determine whether the design of subsequent deep wells needs to be varied. For instance, the arisings might reveal an unexpected layer of impermeable soil within the wetted depth. However, do not ignore the fact that information gleaned from well boreholes is rarely of as high quality as that obtained from dedicated site investigation holes. Any judgments made based on observing the well borehole arisings can only be somewhat speculative.

Whatever drilling technique is used, it is essential that wells are relatively straight and plumb; otherwise, there will be severe operational problems with the submersible pumps. Water supply wells are typically required to be drilled to a verticality tolerance of 1 in 300. This tolerance is probably unnecessary for temporary works wells. Verticality requirements are often not explicitly specified for temporary works wells, but if they are, then 1 in 100 appears to be a more reasonable requirement.

10.5.1 Cable tool percussion drilling

The original cable percussion drilling rig made use of a reciprocating mechanism known as a “spudding arm” or “walking beam” to repeatedly lift and lower a bit (or chisel) suspended on the end of a wire rope inside telescoping sizes of temporary boring casings. The action of the bit breaks up the soil or rock at the base of the borehole to form a slurry with the groundwater entering the well (Figure 10.5a). As the slurry builds up, it deadens the percussive action of the bit; the bore has to be bailed periodically to remove the slurry. The largest rigs were powerful, and progress could be made, albeit slowly, through even boulder and rock formations. Inevitably, the arisings are not much better than a mashed-up slurry of mainly indistinguishable consistency. This type of rig is still in use by some drillers, and is especially prevalent in developing countries.

Sometime later, there came a variation on this technique—the early tripod bored piling rig, which also made use of temporary boring casings of telescoping sizes. The cable passes over a pulley at the top of the tripod and is raised and lowered by a winch linked by a clutch to a small donkey engine. To make progress through granular formations, a sand pump or shell is used, and to penetrate a clay stratum, a clay cutter is used; chisels are used if cobbles or boulders are encountered. Tripod rigs became more mobile and towable. Self-erecting tripod rigs are commonly used for site investigation in the United Kingdom, where they are colloquially known as “shell and auger” rigs, although they are more correctly known as light cable percussion rigs.

Although light cable percussion rigs are most commonly used for 150–200 mm diameter site investigation boreholes, the larger models (of 2- and 3-tonne winch capacity) are widely used to drill temporary works wells. Figure 10.5b shows a 2-tonne rig being used to drill a 300-mm-diameter well bore.

Whatever type of cable tool percussion rig is used, drilling will generally be slower than by the rotary method. This is mainly due to the time required to install and remove the temporary casings. The cable tool percussion method is most suitable for drift deposits such as sand, gravel, and clay. Progress can be very slow indeed if used to drill rock formations, although the method has been successfully used in some soft rocks such as weathered chalk or highly weathered Triassic sandstone.

When using temporary boring casing to penetrate a multilayered soil structure, there is a risk of smearing the side of the hole as the casing is installed and withdrawn. This could mask or block the permeable strata, especially if the individual soil layers are thin. Moreover, when using percussion boring methods, there is some risk that the boring cuttings and slurry will clog the water-bearing formation; thus, subsequent well development should be carefully monitored and supervised (see Section 10.7).

10.5.2 Wash boring and/or jet drilling

The technique of jetting was first developed for the installation of wellpoints, relatively small diameter holes (see Chapter 9), and has been extended to the making of larger holes for the installation of wells and piles.

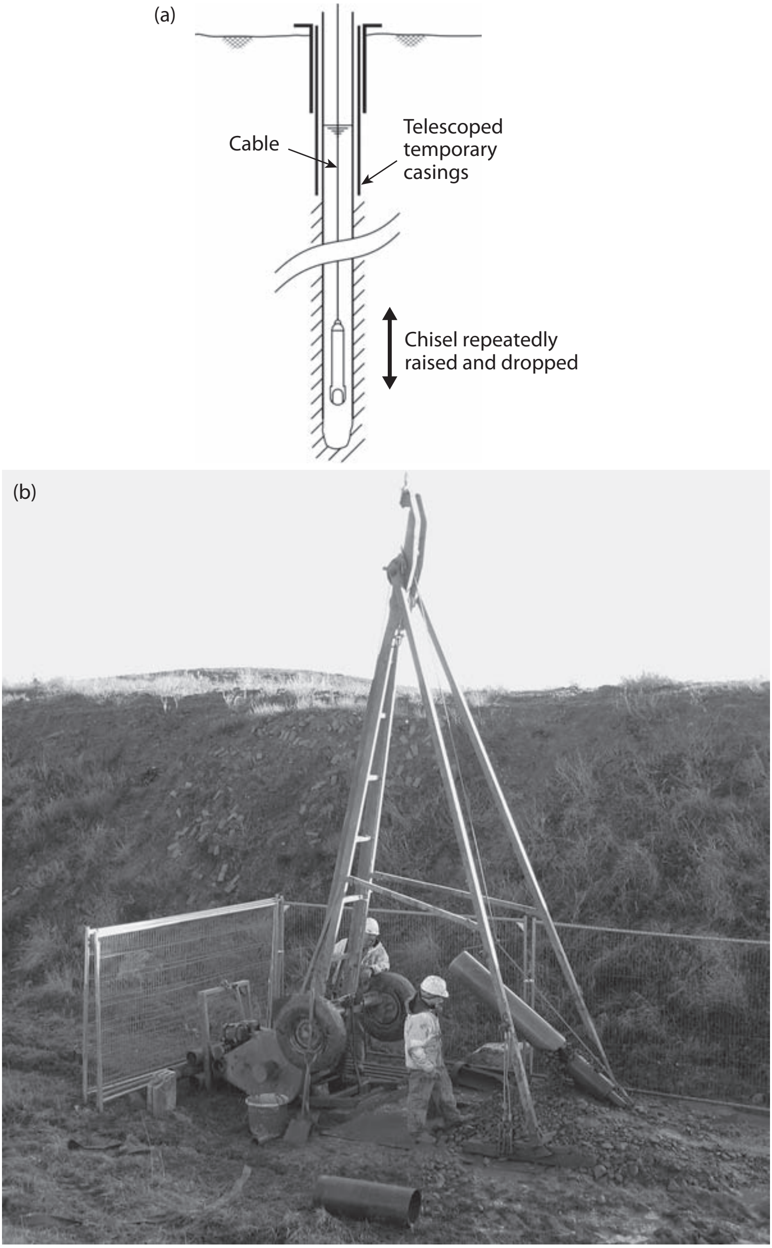


Figure 10.5 Light cable percussion boring. (a) Cable percussion boring method. (b) Light cable percussion boring rig.

Essentially, it entails the use of a jet of high-pressure water being applied at the bottom of the wash pipe or drill pipe such that the soil at the level of the water jet is slurrified (i.e., put into an almost-liquid state). The soil particles tend to be flushed back up to ground level, leaving a bored hole that has been washed relatively clean with little side smear, as might result from cable percussion drilling.

One of the most common applications of this method is the use of a holepuncher or a heavy-duty placing tube used to jet wells into place.

Figure 10.6 Holepuncher used for installation of deep wells. (Courtesy of T.O.L. Roberts.)

Figure 10.6 shows a 300-mm-diameter holepuncher in use. Holepunchers have been used up to diameters of 600 mm and depths of 36 m, although depths of more than 20 m are uncommon.

10.5.3 Rotary drilling: Direct circulation

This method uses a drill bit that is attached to the bottom end of a rotating string of a hollow drill pipe (sometimes known as drill “rods”). Bits are typically a tricone rock-roller bit consisting of movable cutters or fixed drag bits; the action of the bit breaks up the soil or rock in the base of the bore into small cuttings. A continuous supply of drilling fluid is pumped down the drill pipe to cool the rotating bit. The fluid rises back to surface in the annulus between the drill pie and the borehole walls and flushes the cuttings to the surface (Figure 10.7a). Hence, the drill fluid must have sufficient “body” and rising velocity to retain the cuttings in suspension throughout their upward travel to the surface, and also to provide a degree of support to the borehole to prevent collapse.

The bit and the drilling fluid are each vital components to the process, and an understanding of their interdependent functions is necessary to successful usage of the process (Sterrett 2008). The drilling fluid may be water based, with long-chain polymer compounds added to the water to form a fluid with better support and cutting transportation properties. Water-based drilling fluids are sometimes known as “muds”—a term that dates from when bentonite slurries were used to drill wells; bentonite is now rarely used in drilling fluids for water wells. If air (supplied by a powerful compressor) is the drilling fluid, small amounts of additive may be used to form a “foam” or “mist” to carry the cuttings out of the bore. Many modern drilling additives are naturally biodegradable so that any residues left in the ground after development will decay and are less likely to act as an impediment to flow of water into the well.

Upon reaching the required depth of bore, the drill string and bit are withdrawn, with the borehole left filled with drilling fluid prior to installation of the well screen and filter pack. If possible, it is good practice prior to installation to try to displace the fluid in the borehole and replace with a clean, thin solution of fluid.

Rotary drilling rigs are available in a variety of sizes, from small site investigation units that can be towed behind a four-wheel drive vehicle to rigs that are suitable for shallow oil wells that need to be delivered on several articulated trucks and have to be assembled by crane. For groundwater lowering deep wells, self-contained rotary rigs mounted on trucks or on crawler or four-wheel-drive tractors (Figure 10.7b) are most commonly used. Such rigs have drilled wells to depths in excess of 100 m.

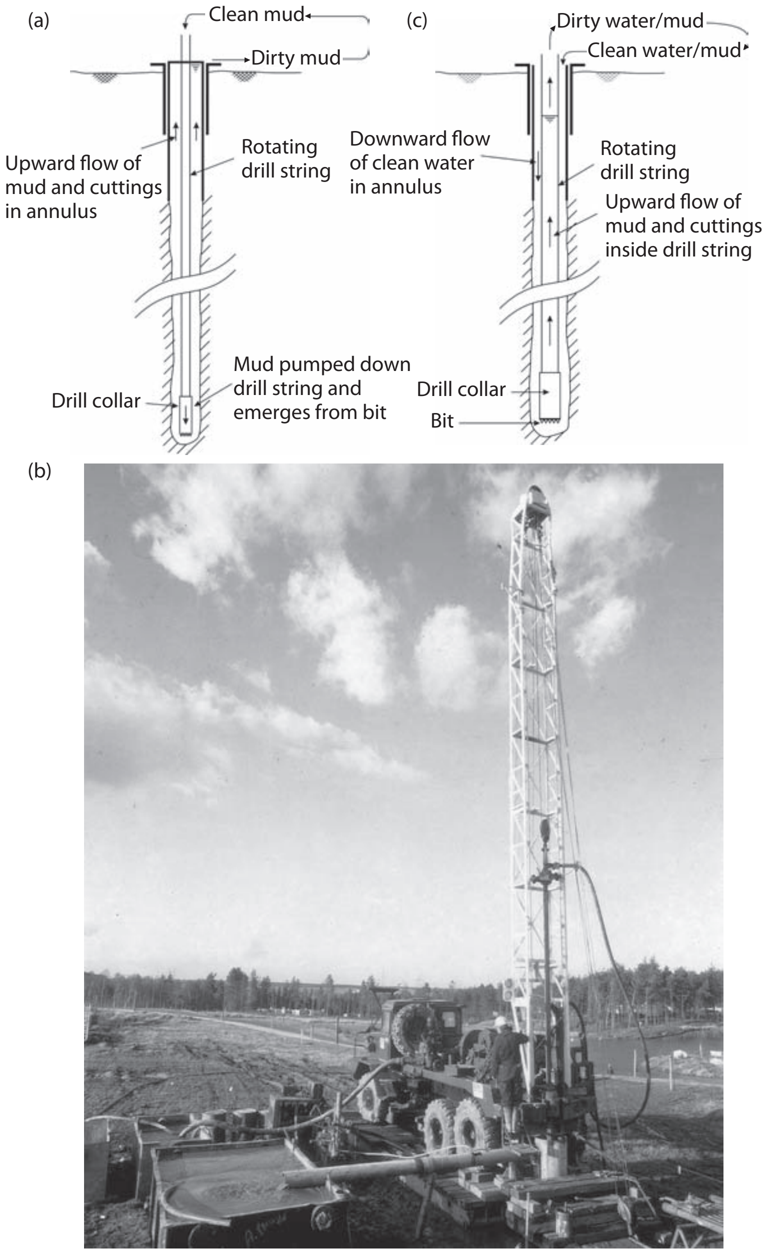


Figure 10.7 Rotary drilling. (a) Direct circulation rotary drilling method. (b) Truck-mounted direct circulation rotary drilling rig. (Courtesy of British Drilling and Freezing Company Limited, Nottingham, U.K.) (c) Reverse circulation rotary drilling method.

10.5.4 Rotary drilling: Reverse circulation

This is a variation of rotary drilling that is often used to drill large-diameter water supply wells but is used only rarely to construct temporary works deep wells. The principle of operation is the same as for direct circulation rotary drilling, but the direction of flow of drilling fluid is reversed (Figure 10.7c).

The mixture of drilling fluid and cuttings in suspension flows up the hollow drill pipe with a fast rising velocity, because the internal diameter of the drill pipe is limited. The borehole is kept topped up with drilling fluid, which allows the return flow of fluid to descend slowly in the annulus—of greater cross-sectional area than the drill pipe—between the drill pipe and the borehole wall. The level of the drilling fluid must always be a few meters above the level of the groundwater to help maintain a stable bore. As with the direct circulation method, upon reaching the required depth of bore, the drill string and bit are withdrawn, and the borehole is kept topped up with fluid.

10.6 INSTALLATION OF WELL MATERIALS

Once the bore is complete, the well casing and well screen are placed in the hole. The casing and screen consist of threaded pipe, typically between 150 and 300 mm in diameter, supplied in lengths of between 1 and 6 m. The diameter of casing and screen required is determined as described in Section 10.3. The casing sections are plain (unperforated), and the screen sections are perforated (generally by slotting) and, for temporary works wells, are typically made from uPVC or HDPE; steel screen and casing are used only rarely (see Section 10.3.3).

Prior to installation, the drilling crew should be instructed on the number and sequence of casing and screen lengths to be installed to ensure that the screens are located within the water-bearing horizons. The screen and casing is installed in sections by lowering into the borehole using the drilling rig's winch. A bottom cap is fitted to the first length of screen or casing that is lowered into the bore; further sections are then added until the string of screen and casing is installed to the required depth. It is often a good idea to place a few hundred millimeters of filter gravel in the base of the bore before commencing installation of screen and casing. This prevents the bottom length from sinking into any soft sediment or drilling residue remaining in the base of the hole.

When the screen and casing are in position, the filter pack (see Section 10.3.4) must be placed in the annulus between the well screen and the borehole wall. The filter pack is formed from filter media generally consisting of uniformly sized gravels, although coarse sands are sometimes used. The filter material may be supplied in bags or in bulk loads. Ideally, to ensure correct placement, the gravel should be installed using one or more tremie pipes. The tremie pipe would typically be of uPVC or steel sectional pipe of 50-mm internal bore (although sizes down to 32-mm bore have been used to place uniform sands). The filter media are poured slowly into a hopper at the top of the tremie, perhaps washed down by a gentle flow of water. Patience is vital in this operation; adding the filter media too quickly will cause the tremie pipe to block or "bridge." If this occurs, the tube will have to be removed and flushed clear, causing delays and inconvenience. The tremie tube is raised slowly, keeping pace with the rising level of filter media.

It is sometimes acceptable to place the filter media without a tremie pipe by pouring into the annulus from the surface. This is only acceptable if the filter medium is very uniform and will not segregate as it falls down the bore and if the wall of the borehole is very stable (e.g., if supported by temporary casing). Again, care must be taken to ensure that the filter media are added slowly. If too much is added at once, a blockage or "bridge" may occur; this can be difficult to clear and may result in the screen and casing having to be removed and the complete installation recommenced.

If temporary drill casing is used to support the borehole, this must be removed in sections as the filter medium is added. The level of the filter media should not be allowed to rise significantly above the base of the temporary casing; otherwise, a "sand lock" may occur between the temporary drill casing and the well screen. This can cause the well screen to be pulled out when the temporary casing is withdrawn. The best practice is to alternately add some filter media, pull the temporary drill casing out a little, add more filter media, pull the drill casing further, and so on, monitoring the level of the filter media continuously. Boreholes drilled by rotary methods without temporary casing do not have this constraint, and filter media can be placed in one continuous, steady operation.

The filter pack is sometimes brought up to ground level, but increasingly, it is good practice to place a very low-permeability grout seal above the filter pack (see Figure 10.2). This reduces the risk of aquifer contamination by surface water (or water from other aquifers) passing down the filter pack into the aquifer (see Section 15.5). The grout may be neat cement or cement–bentonite (see the work of Sterrett 2008 for guidance on grout mixes and placement). If the grout was placed directly on top of the filter pack, some grout may be lost into the filter media, compromising its permeability. To avoid this, a 1–2 m thick layer of bentonite pellets should be placed on top of the filter pack, and allowed to swell before placing the grout.

10.7 WELL DEVELOPMENT

Development is a process, carried out between completion of the well and installation of the pumps, with the aim of removing any drilling residue or debris from the well and maximizing the yield of clean sand-free water. If development is not carried out, not only might the yield be low but also the borehole electro-submersible pump will be damaged as a result of pumping sand-laden water.

The most commonly used forms of development in granular aquifers are intended to induce two-way groundwater flow between the well and the aquifer to remove any loose particles from the filter pack and aquifer immediately around the well. This will increase the aquifer permeability locally and will remove any potentially mobile soil particles that might damage the operational pumps. It is most important that the development generates a two-way flow (alternately into and out of the well) to help dislodge mobile soil particles that may be loosely wedged in soil pores—this is much more effective than continuous pumping. Development can only be effective in wells that have appropriately designed and installed filter packs. No amount of development can correct a well with a filter pack that is too coarse or is discontinuous due to installation problems.

There is a wide range of development techniques (Sterrett 2008) used on water supply wells, but in practice, most temporary works wells are developed by one of the following three techniques:

1. *Airlift*. Air from a compressor is used to lift water from the well up an eductor tube formed from plastic or steel pipe of 75–150 mm diameter (Figure 10.8a). This is a robust method of pumping and, with suitable equipment, can transport significant volumes of sand with the water. Reversal of flow is achieved by lowering the air line past the bottom of the eductor tube and delivering a short blast of air into the well. Alternatively, airlift pumping can be used for a few seconds to raise a "slug" of water to just below ground level, at which point the air is turned off, causing the water to fall back down the well and inducing flow out of the well and into the aquifer. The airlift method is most effective when the water level in the well is near the surface and becomes less efficient for deeper water levels.
2. *Surge block*. A tight-fitting block is lowered into the well screen (Figure 10.8b) and pulled sharply upward using a tripod and winch (such as a light cable tool percussion rig). As the block moves upward, water will be forced out of the well above the block and drawn in below, thereby achieving reversal of flow. There are a number of ways that the block can be surged using either many short strokes or fewer long ones (Sterrett 2008); some methods of surging are known as "swabbing the well." If a special surge block is not available, a weighted shell or bailer of slightly smaller diameter than the well screen is sometimes used. The sediment and debris that builds up in the well will need to be removed by airlift pumping or bailing.

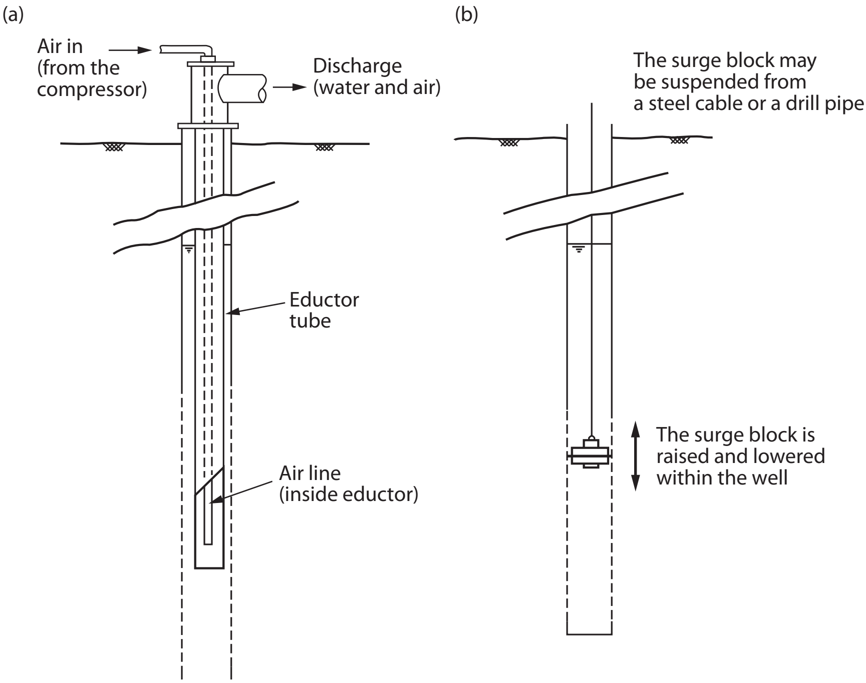


Figure 10.8 Well development methods. (a) Airlift with an eductor tube. (b) Surge block.

3. *Jetting.* Less commonly used, this method involves lowering a jetting head (mounted on drill pipe) down inside the well screen. The drill pipe and jetting head are slowly rotated, and high-pressure water is pumped down the drill pipe and jets horizontally at the screen via small nozzles in the head. The jetting generally forces flow into the aquifer and may therefore need to be alternated with airlift pumping to get flow reversal and also to remove any sediment or debris generated by jetting.

Development is usually discontinued when the well no longer yields sand or fine particles when pumped by the airlift. This is normally monitored by observing the discharge water, which initially will appear very dirty or discolored, but will become clear as development proceeds. However, a note of caution should be sounded. Sometimes, the discharge water may appear clear but still contains a small but significant amount of fine sand, which is enough to cause problems for the pump and to create voids around the well. The best way to check for this is to take a sample of water in a clean white plastic tub (of the sort used to hold soil samples). Any sand will be clearly visible in the bottom of the tub. A specialist sediment sampling container called an “Imhoff cone” can also be used to check for sediment in the water, but this device is intended for much higher sediment loads, and in many ways, inspection of a sample in a white tub is a more sensitive method.

The question is often asked: How long should it take to develop a well? There is no simple answer to this, but experience suggests that many wells in granular aquifers will take between 6 and 12 h to be effectively developed. Certainly, if a well still yields copious amounts of sand after more than 2–3 days (for example, 20–30 h of development), the well is unlikely to improve, and a replacement well (possibly with a different filter medium) should be considered.

For wells drilled in carbonate rocks, such as chalk, wells may be developed by acidization. This is a technique commonly used in water supply wells (Banks et al. 1993), where acid is introduced into the well to dissolve any drilling slurry and to improve the well–aquifer connection by dissolving aquifer material in fissures locally around the well. In practice, several tonnes of concentrated acid are introduced into the well. The reaction between the acid and the carbonate rock generates large volumes of carbon dioxide gas, which must be carefully controlled by fitting a gas-tight head plate (equipped with specialist valves and pressure relief devices) to the top of the well. Acidization is not a straightforward procedure and should be planned and undertaken by experienced personnel, with particular emphasis on ensuring that necessary health and safety measures are in place. Acidization is not commonly used to develop temporary works wells, but the method has been used on some deep groundwater lowering wells into the chalk beneath London, including the Jubilee Line Extension Tunnel Project in the 1990s.

If wells are operated for long periods of time (several months or years) and are affected by encrustation or clogging, well performance may deteriorate. In such cases, well performance may be improved by periodic redevelopment using one of the methods described above (including acidization, which has been used on heavily encrusted wells).

10.8 INSTALLATION AND OPERATION OF DEEP WELL PUMPS

Following development, the pumps can be installed. The following sections deal exclusively with the procedures used with borehole electro-submersible pumps (see Section 13.5), the most commonly used type in deep well systems.

10.8.1 Installation of borehole electro-submersible pumps

The pump is connected to the riser pipe, a steel or plastic pipe that carries the discharge from the pump up to ground level (see Figure 10.2). The riser pipe is typically between 50 and 150 mm in diameter for the smallest and largest pumps used for temporary works. The riser is supplied in sections with threaded or flanged joints; extra sections are added until the pump has been lowered to the design level (see Figure 13.6). The electrical power cable that is connected to the pump is also paid out as the pump is lowered into position. The cable should be kept reasonably taut and be taped or tied to the riser pipe every few meters. If this is not done, then any slack in the cable may form “loops” down the well, interfering with the subsequent installation of dip tubes.

If steel riser pipe is used, it may be strong enough to support the weight of the pump, riser pipe, and water column. Otherwise (and, in any case, if a plastic riser pipe is used), the weight must be supported by straining cables or ropes from the pump, tied off at the wellhead. It is good practice to install a dip tube of 19–50 mm diameter to allow access for a dipmeter to be used to monitor the water level in the well.

For groundwater lowering applications, the pump is normally installed near the base of the well so that potential drawdown is maximized. The base of the pump should be at least 1 m above the base of the well to avoid the pump becoming stuck in any sediment that may build up at the bottom of the well. When the pump is installed, a headworks arrangement—typically including a control valve and pressure gauge—is attached to the top of the riser pipe and connected to the discharge pipework.

The best sort of valve for use at the wellhead is the gate valve. It is preferable because its slide plate is at right angles to the water flow and its position can be finely adjusted by its screw mechanism to regulate the flow. Its sensitivity of opening adjustment is good, and when fully open, it offers less resistance to flow than any other type of valve. The other type of valve sometimes used for well output control is the butterfly valve. It is less costly than the gate valve (size for size), but the sensitivity of opening control is less.

Most borehole electro-submersible pumps are powered by a three-phase supply; if the phases are connected in the wrong order, the pumps can run (or rotate) backwards (i.e., anticlockwise rather than clockwise). Pumps running backwards move very little water; thus, on installation, it is important to “check the direction” of each pump in turn. One method is to start the pump with the control valve fully closed and observe the pressure gauge. If the needle indicates a high pressure (consult the pump manufacturer’s data sheets for the expected value), the motor is running in the right direction. However, if the indicated pressure is low, the motor may be running backward. Turn the pump off, isolate the pump switchgear, have an electrically competent person change over two of the phase wire connections, and restart the pump against a closed valve. The pressure gauge should register full pressure, indicating that the motor is now running in the right direction.

10.8.2 Adjustment of electro-submersible pumps

For groundwater lowering projects, the pumps generally run continuously, with the operating level in individual wells only a little above the level of each pump intake. Generally, the flow from each pump is adjusted to achieve this condition by manipulation of the control valve on each wellhead. Partial closure of the valve will impose just sufficient back pressure (extra artificial head) to constrain the well output and maintain the optimum operating level.

If the well operating level is too close to the level of the pump intake, the pump will tend to "go on air" from time to time; this is when the water level in the well reaches the pump intake, and air is drawn into the pump. In this condition, the pump will race when "on air," and the riser pipe will tend to vibrate. This must not be allowed to continue in the long term, because under such conditions, the pump motor will be damaged severely. The symptoms of "going on air" can be detected by observing the needle of the pressure gauge at the wellhead sited upstream of the control valve (see Figure 10.2) and by observing the vibration of the riser pipe. When the discharge flow contains some air (rather than water only), the gauge needle will repeatedly drop to zero. If these erratic variations in pressure gauge readings are observed, close the control valve gradually until the pressure gauge needle remains steady; a very slight fluctuation in the needle position is acceptable. Alternatively, if the gauge needle remains steady, the well operating level may be above the design level. If this condition is suspected, gently open the control valve until the needle starts to flicker and then close the control valve by a very small amount.

10.8.3 Use of deep wells on tidal sites

On sites where groundwater levels vary tidally, the pumped level in each well may vary too; although generally, these variations will be small (less than a few meters). It is suggested that the control valve is adjusted for the low-tide condition only and not altered unless the amount of lowering required at the high tide condition is critical. If this is the case, frequent adjustment of all operating levels will be required. This is tedious and liable to be overlooked on occasions, especially during night shifts. If the maximum lowering is required at high and low tides, one possible solution is the installation of additional wells to be operated only for a few hours on either side of a high tide.

10.8.4 Use of oversized pumps

Occasionally, it may not be possible to equip a well with the appropriate output pump, and so, a pump rated at an output greater than necessary is installed in the well. Oftentimes, in such circumstances, the lack of sensitivity on the control valve will make it difficult (sometimes even impossible) to adjust the well output so that continuous pumping is practicable. There are two alternative expedients that can be employed in such circumstances:

1. At the wellhead, fit an additional small bore pipe and valve to bypass the main control valve. Operation involves closing the small bypass valve completely and adjusting the main valve to achieve the best possible coarse adjustment and then gradually opening the bypass valve to make the best fine adjustment. This mode of operation makes feasible continuous pumping.
2. Modify the pump controls for intermittent running with automatic stop/start. This requires sensing units in the pump electrical controls linked to upper and lower level sensor electrodes in the well.

10.8.5 Level control switches

If the pumps are to be operated intermittently, sensor electrodes in the well are used to trigger the pump to start when the water level is high and stop when the water level is too low. Typically, the lower level electrode is set to switch the pump motor off just before the water level is drawn down sufficiently to cause the pump to "go on air." The upper sensor is set to switch the pump on when the water level in the well has recovered to a predetermined level (which must obviously be below excavation formation level). The average level of lowering achievable by this mode of pumping will be approximately the mean level between the two sensors. The distance between the upper and lower sensor electrodes should be the minimum practicable to allow the best possible amount of lowering by intermittent pumping. The disadvantage of this is that it will cause more frequent starts of the pump motor, increasing the risk of motor failure.

10.8.6 Problems due to frequent start-up of motor

Most borehole electro-submersible pumps are operated using direct-on-line electrical starter controls. With this arrangement, the starting current of an electric submersible pump motor is greater than the running current by around a factor of up to six. The heat generated on start-up is significant. Thus, if the motor is frequently switched on and off (e.g., due to the use of level controls), with time, the motor windings may tend to become overheated, eventually resulting in the failure of the motor. There is a risk of this happening if there is on/off switching more than approximately six times per hour. In fact, even six starts per hour is a lot, and the number of starts should be minimized if at all possible.

10.8.7 Encrustation and corrosion

Deep wells operating for long periods may sometimes be affected by encrustation due to chemical precipitation or bacterial growth in the well screen, filter pack, pump, or pipework. Encrustation may reduce the efficiency of the well (reducing yield and increasing well losses) and may increase the stress on the pump, leading to more severe wear and tear and, ultimately, failure.

In certain types of groundwater, corrosion of metal components in pumps and pipework may be severe. Recognition of such conditions is vital when specifying equipment—for example, maximizing the use of plastic pipework can reduce the problems, only leaving the submersible pumps made from metal.

The nature of these problems, and appropriate avoidance and mitigation measures are discussed in Section 16.9.

10.9 VACUUM DEEP WELL INSTALLATIONS

If deep wells are installed in a low-permeability aquifer and well yields are low, well performance can be enhanced somewhat by sealing the top of the well casing and applying a vacuum. The vacuum is generated by an exhauster unit (a small vacuum pump) located at ground level. Typically, the exhauster unit would be connected to a manifold, allowing it to apply vacuum to several wells. Because airflow to the exhauster will be low (once vacuum is established), the pipework connecting the exhauster and the wells need only be of small diameter, perhaps 50 mm or less. For the method to be effective, the top of the well filter pack must have a bentonite or grout seal; otherwise, air may be drawn through the gravel pack into the well, preventing the establishment of a vacuum. A typical vacuum deep well is shown in Figure 10.9.

The top of the well needs to be sealed airtight between the well casing and the pump riser pipe, power cable, and vacuum supply pipe. Purpose-built wellhead seals are available, but plywood disks and copious amounts of sealing tape have also proved effective.

The application of vacuum to deep wells is unlikely to be a panacea for a poorly performing system, especially if poor yields are associated with inappropriate filter gravel gradings or inadequate well development. The yield from the wells will not be transformed wholesale, but, at best, will increase by 10%–15%, occasionally more. Even if a vacuum equivalent to 8 m head of water can be maintained in the wells, do not expect the drawdown outside the wells to increase by a similar amount. It may be that the vacuum is mainly used to overcome well losses at the face of the well. The application of vacuum can cause operational problems including

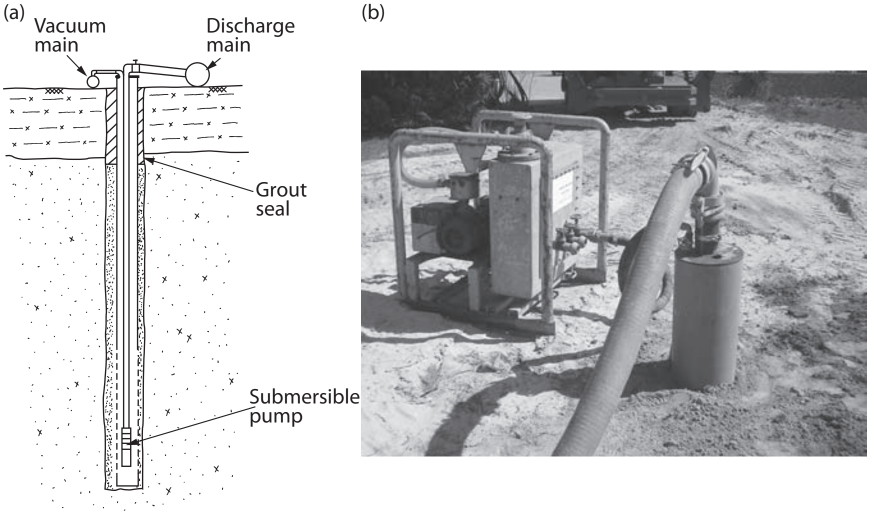


Figure 10.9 Vacuum deep well. (a) Schematic section through a vacuum deep well. (b) Wellhead of a vacuum deep well. The wellhead is sealed, and the exhauster unit (next to the well) is used to generate a vacuum in the well. (Courtesy of Hölscher Wasserbau GmbH, Haren, Germany.)

increased risk of collapse of well screens or sand being pulled through the filter pack.

10.10 SHALLOW WELL INSTALLATIONS

The bored shallow well system is a synthesis of the deep well and wellpoint systems. Bored shallow wells are constructed in the same way as a deep well system, but they use the wellpoint suction pumping technique to abstract the water (Figure 10.10). Hence, the amount of lowering that can be achieved is subject to the same limitation with a wellpoint system; that is, drawdowns in excess of 6 m below pump level are difficult to achieve. The method is most useful on congested urban sites and where the soil permeability is high. Because wells are of larger diameter than wellpoints, they generally have a greater yield and can therefore be at greater centers. This means that the well creates fewer constraints on the activities of the steel fixers, shutter erectors, and other trades needing to carry out work within the dewatered excavation. The shallow well method is known by some as "jumbo" wellpointing, because the wells can be thought of as widely spaced, grossly oversized wellpoints.

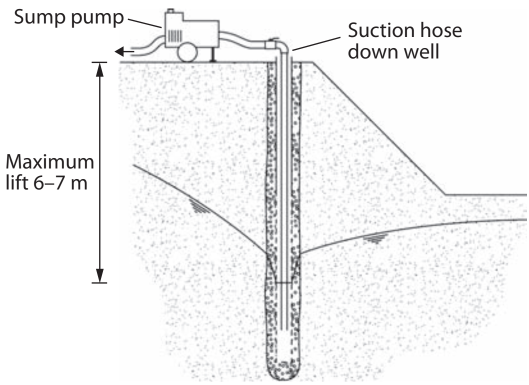


Figure 10.10 Shallow well system. (From Preene, M. et al., Groundwater control—design and practice, Construction Industry Research and Information Association, *CIRIA Report C515*. London, 2000. Reproduced by kind permission of CIRIA: www.ciria.org.)

10.11 CASE HISTORY: TEES BARRAGE, STOCKTON-ON-TEES

In the early 1990s, a barrage was constructed across the River Tees at a site between Stockton-on-Tees and Middlesbrough (Leiper and Capps 1993; Franklin and Capps 1995). The barrage was constructed by Tarmac Construction Limited on behalf of the Teeside Development Corporation. The barrage substructure (a reinforced concrete slab 70 m wide, 35 m long, and 5 m thick) was formed in a large construction basin (Figure 10.11), which provided dry working conditions while the river flow was diverted to one side during the works.

The construction basin was approximately 70 m × 150 m in plan dimensions and was contained within bunds containing sheet-pile cutoff walls; the formation level of the basin was at –8 mOD. The bunds were formed "wet" across the river, and it was intended that the water that remained trapped in the basin when the bunds were closed would be removed by sump pumping. Ground investigations indicated the soil in the floor of the basin to be very stiff glacial clay, an ideal material on which to found the barrage. However, a few meters below the floor of the basin, a confined aquifer of glacial sand and gravel was present (see Figure 10.12), with a mean piezometric level of +1 to 2 mOD (up to 10 m above the floor of the excavation). The high piezometric level in this aquifer (and the possible presence of gravel lenses in the glacial clay) meant that, when the basin was pumped dry, if the piezometric pressure was not lowered significantly, there would be a risk of heave of the base of the excavation.

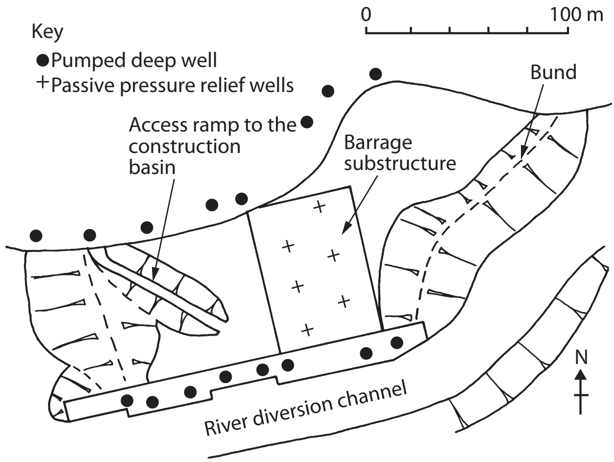


Figure 10.11 Schematic plan of the construction basin for Tees Barrage. Deep wells are located to the north and south of the basin. Pressure relief wells are located in the deepest part of the basin. (After Leiper, Q.J. and Capps, C.T.F., *Engineered Fills*, Clarke, B.G., Jones, C.J.F.P., and Moffat, A.I.B., eds., Thomas Telford, London, 1993, pp. 482–491.)

The solution that was adopted was to install a system of deep wells around the perimeter of the bund to lower the piezometric level down to formation level and ensure that factors of safety against heave were acceptably high. Because there was limited time available for additional ground investigation when the problem was identified, the system design was finalized using the observational method (see Section 7.3).

Two wells were installed and test pumped (by constant rate pumping and recovery tests) in turn, with the other well being used as an observation well. Although these wells were intended as trial or test wells, they

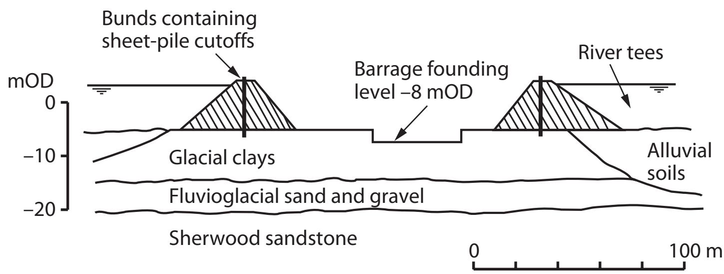


Figure 10.12 Section through the construction basin for Tees Barrage showing ground conditions. (After Leiper, Q.J. and Capps, C.T.F., *Engineered Fills*, Clarke, B.G., Jones, C.J.F.P., and Moffat, A.I.B., eds., Thomas Telford, London, 1993, pp. 482–491.)

were located carefully so that, if the trial was successful, the wells could be incorporated into the final deep well system. The trial allowed flow rate and drawdown data to be gathered, and the system was designed using the cumulative drawdown method (see Section 7.7). A particular feature of the test data was that, when pumping was interrupted, piezometric pressures recovered rapidly to close to their original levels. This is a characteristic feature of confined aquifers and means that any design should strive to ensure that any breakdowns or interruptions in pumping are minimized.

The final pumped system employed had the following key elements, several of which were specified based on the expectation of the rapid recovery of piezometric levels:

1. A system of 16 deep wells was installed to abstract from the sand and gravel aquifer. This included an additional two wells over the minimum number required to allow for maintenance or individual pump failure. The wells were drilled by cable percussion methods at 300-mm boring diameter, to allow the installation of 200-mm nominal diameter well screen and casing, which in turn allowed the installation of electro-submersible pumps of 10-L/s nominal capacity.
2. The power supply was split into two separate systems, with each part feeding half the pumps. This was to reduce the risk of a power system failure knocking the whole system out.
3. Standby generators were permanently connected into the system, ready for immediate start-up, and were connected to alarm systems.
4. Twenty-four-hour supervision was provided by the dewatering subcontractor.
5. Passive relief wells were installed in the base of the excavation to provide additional pressure relief capacity in the event of a total system failure. In such an event, the wells would have overflowed and flooded the excavation in a controlled manner, but without the risk of heave at formation level.
6. All piezometers, deep wells, and pressure relief wells were monitored regularly to ensure that the system was operating satisfactorily. A positive management system was established for the monitoring, with a proper inspection record kept by an approved and suitably experienced and qualified individual.

The system produced a total yield of 95 L/s in steady-state pumping (compared with a nominal installed pumping capacity of 16 wells at 10 L/s each) and operated for the 11-month life of the basin. On completion, the pumps were removed, and the deep and relief wells were backfilled with gravel, bentonite, and concrete.

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Chapter 11

Other dewatering systems

11.1 INTRODUCTION

This chapter describes some other less commonly used dewatering and groundwater control systems and outlines conditions appropriate to their use.

The systems described in this chapter are

1. Ejector systems, which are appropriate to low-permeability soils such as very silty sands or silts
2. Horizontal wellpointing, which is mainly suitable for dealing with large trenching or pipeline projects of limited depth
3. Horizontal wells
4. Pressure relief wells
5. Collector wells
6. Siphon drains
7. Electro-osmosis, which is applicable to very low-permeability soils or for increasing the shear strength of very soft soils
8. Artificial recharge systems
9. Applications of dewatering and groundwater control technologies used for the control or remediation of contaminated groundwater

Some of the techniques described are specialized and, perhaps, rather esoteric in nature. An engineer working in groundworks and excavations might spend a whole career in the field and never have to apply any of these techniques. Nevertheless, it is important to be aware of the specialist methods that may be of help when faced with difficult conditions.

11.2 EJECTORS

The ejector system (also known as the eductor or jet-eductor system) is suitable for pore water pressure reduction projects in low-permeability soils such as very silty sand, silt, or clay with permeable fabric. In such soils, the total flow rate will be small, and some form of vacuum assistance to aid drainage is beneficial (see Section 5.5); the characteristics of ejectors are an ideal match for these requirements.

Essentially, the ejector system involves an array of wells (which may be closely spaced like wellpoints or widely spaced like deep wells), with each well pumped by a jet pump known as an ejector. Ejector dewatering was developed in North America in the 1950s and 1960s, when jet pumps used in domestic supply wells were first applied to groundwater lowering problems. Since then, the technique has been applied in Europe, the Far East, and the former Soviet Union. Ejectors were not widely employed in the United Kingdom before the late 1980s; the A55 Conwy Crossing Project was one of the first U.K. projects to make large-scale use of ejectors (Powrie and Roberts 1990). Some applications of the ejector method in the U.K. are described by Preene and Powrie (1994) and Preene (1996).

11.2.1 Merits of ejector systems

The ejector system works by circulating high-pressure water (from a tank and supply pumps at ground level) down riser pipes and through a small-diameter nozzle and venturi located in the ejector in each well. The water passes through the nozzle at high velocity, thereby creating a zone of low pressure and generating a vacuum of up to 9.5 m of water at the level of the ejector. The vacuum draws groundwater into the well through the well screen, where it joins the water passing through the nozzle, piped back to ground level via a return riser pipe, and then back to the supply pump for recirculation. A schematic ejector well system is shown in Figure 11.1. Two header mains are needed. A supply main feeds high-pressure water to each ejector well, and a return main collects the water coming out of the ejectors (consisting of the supply water plus the groundwater drawn into the well). This large amount of pipework is needed to allow the recirculation process to continue.

The most obvious advantage of an ejector system is that ejectors will pump both air and water. As a result, if the ejectors are installed in a sealed well in low-permeability soil, a vacuum will be developed in the well. This is one of the main reasons that ejectors are suitable for use in low-permeability soils, where the vacuum is needed to enhance drainage of soils into the wells. Another advantage is that the method is not constrained by the same suction lift limit as a wellpoint system (see Chapter 9). Drawdowns of 20–30 m below the pump level can be achieved with commonly available equipment, and drawdowns in excess of 50 m have been achieved with systems capable of operating at higher supply pressures.

These characteristics mean that ejector systems are generally applied in one of the following two ways:

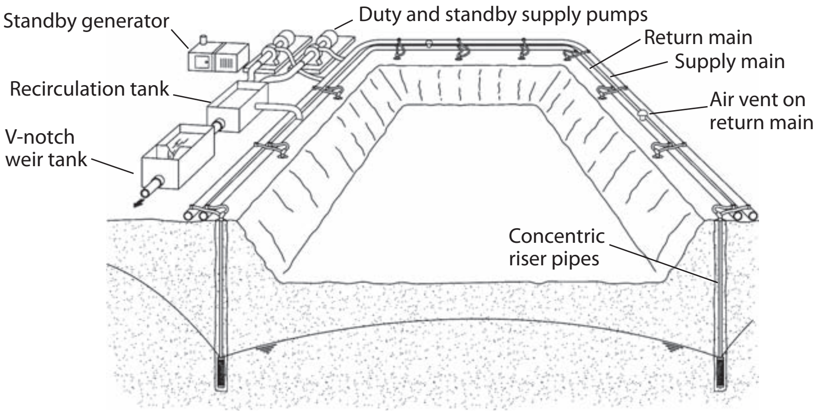


Figure 11.1 Ejector system components. (From Preene, M. et al., Groundwater control—design and practice, Construction Industry Research and Information Association, *CIRIA Report C515*, London, 2000. Reproduced by kind permission of CIRIA: www.ciria.org.)

1. As a vacuum-assisted pore water pressure control method in low-permeability soils.
2. As a form of "deep wellpoint" in soils of moderate permeability as an alternative to a two-stage wellpoint system or a low-flow-rate deep well system.

It is also important to be aware of some of the practical limitations and drawbacks of the ejector system. Perhaps, the most significant drawback is the low mechanical (or energy) efficiency of ejector systems. In low- to moderate-permeability soils, where flow rates are small, this may not be a major issue, but in higher-permeability soils, the power consumption and energy costs may be huge in comparison to other methods. This is probably the main reason that the ejector system is rarely used in soils of high permeability. Another potential problem is that, due to the high water velocities through the nozzle, ejector systems may be prone to gradual loss of performance due to nozzle wear or clogging. This can often be mitigated by regular monitoring and maintenance, but it may make long-term operation less straightforward.

11.2.2 Types of ejectors

An ejector is a hydraulic device that, despite having no moving parts, acts as a pump. Several different designs of ejectors are available, each having different characteristics. These different ejector designs can be categorized into two types based on the arrangement of the supply and return riser pipes:

1. *Single (or concentric) pipe* (Figure 11.2). This design has the supply and return risers arranged concentrically, with the return riser inside the supply riser. The supply flow passes down the annulus between the pipes, through the ejector, and then returns up the central pipe.
2. *Twin (or dual) pipe* (Figure 11.3). Here, the supply and return risers are separate, typically being installed parallel to each other.

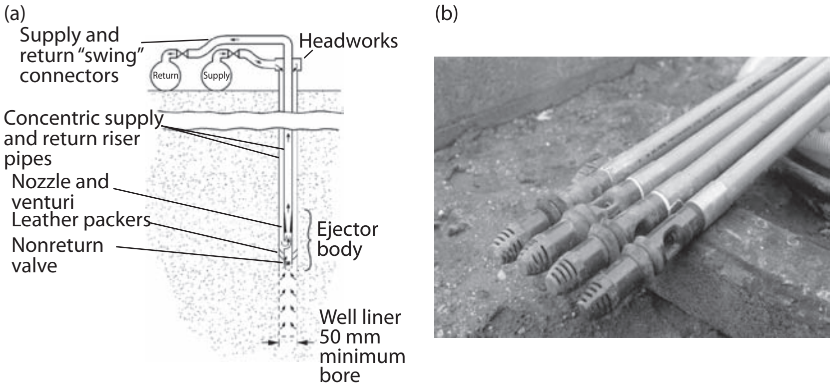


Figure 11.2 Single-pipe ejectors. (a) Schematic view. (From Preene, M. et al., Groundwater control—design and practice, Construction Industry Research and Information Association, *CIRIA Report C515*, London, 2000. Reproduced by kind permission of CIRIA: www.ciria.org.) (b) Ejectors attached to central riser pipes ready for installation into the well. (Courtesy of WJ Groundwater Limited, Bushey, U.K.)

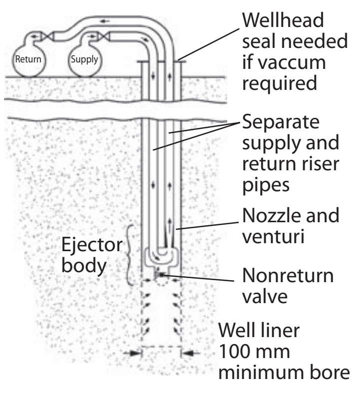


Figure 11.3 Twin-pipe ejectors—schematic view. (From Preene, M. et al., Groundwater control—design and practice, Construction Industry Research and Information Association, *CIRIA Report C515*, London, 2000. Reproduced by kind permission of CIRIA: www.ciria.org.)

Both types are used in dewatering systems. The single-pipe ejector has the advantage that the outer pipe can also be used as the well casing, provided that it has sufficient pressure rating; this allows ejectors to be installed in well casings of 50-mm internal diameter. Twin-pipe ejectors need to be accommodated in rather larger well casings (approximately 100-mm internal diameter). The installation and connection of twin-pipe ejectors involves rather simpler plumbing than for single-pipe designs; this can make the twin-pipe type more suitable for use in localities where skilled labor is scarce, and when it is desired to keep dewatering equipment as simple as practicable.

11.2.3 Installation techniques

The ejectors themselves (the jet pumps) are installed in wells, which are generally installed by similar methods to deep wells: cable percussion boring, rotary drilling, jetting, or auger boring (see Chapter 10). Well casings and screens and filter packs are then installed in the borehole. If single-pipe ejectors are used, the smaller diameter casings are sometimes installed by methods more akin to wellpointing (see Chapter 9) than deep wells. Ejector wells will normally need to be developed following installation and prior to placement of the ejectors and risers.

The ejectors are connected to the supply and return riser pipes (twin or single pipe, depending on the design) and are lowered down to the intended level, typically near the base of the well. The headworks are fitted to seal the top of the riser pipes to the well liner, and the flexible connections are made to the supply and return mains (Figure 11.1). This is needed to connect each ejector to the pumping station(s) that supply the high-pressure water, which is the driving force behind the pumping system.

11.2.4 Ejector pumping equipment

The pumping equipment that makes up an ejector system consists of three main parts: (1) the ejector (and associated riser pipes and headworks); (2) the supply pumps; and (3) the supply and return pipework.

Ejectors are a form of pump, and similar to any pump, their output will vary with discharge head. An additional complication is that, in order to function, the ejector must be supplied with a sufficient supply of water at adequate pressure (typically 750–1500 kPa measured at ground level). Each design of ejector will have different operational characteristics; thus, performance curves will be needed for the model being used (Figure 11.4). It is important that the performance curves are representative of the conditions in a well (see the work of Miller 1988 and Powrie and Preene 1994b for further details). At a given supply pressure and ejector setting depth, the pumping capacity (the induced flow rate) can be estimated from the

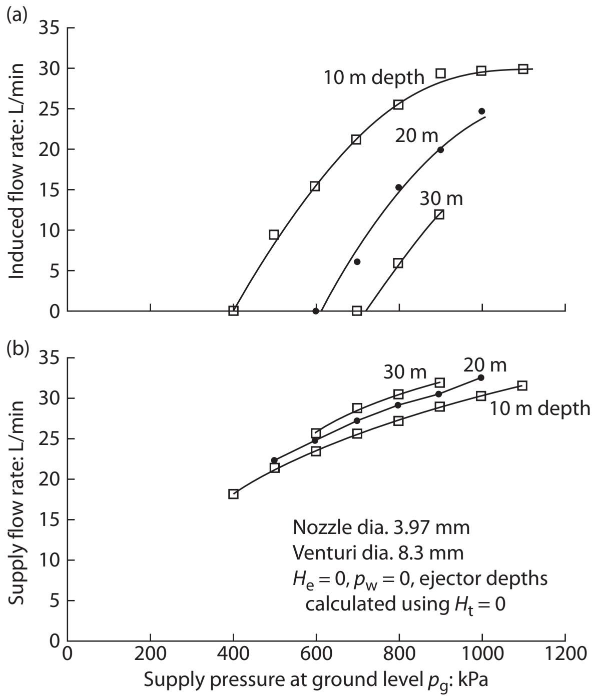


Figure 11.4 Example of ejector performance curves. (a) Graph is used to determine the supply pressure necessary to achieve the desired induced flow rate per ejector at the specified ejector setting depth (10, 20, and 30 m shown). (b) Graph is used to determine the supply flow rate per ejector necessary to maintain the specified supply pressure. (From Powrie, W., and Preene, M., *Proceedings of the Institution of Civil Engineers: Geotechnical Engineering*, 107, 143–154, 1994. With permission.)

performance curves. If this is sufficient to deal with the anticipated well inflow, then the required supply flow per ejector (at the specified pressure) and the number of ejectors to be fed by each pump can be used to select the supply pumps.

If the ejector-induced flow rate is not large enough to deal with the predicted well inflow, it may be possible to increase the capacity of the ejector by increasing the supply pressure. However, Figure 11.4 shows that, as the supply pressure is increased, the induced flow rate of the ejector tends to plateau. This is caused by cavitation in the ejector, and additional increases in supply pressure beyond that point do not give a corresponding increase in ejector capacity. With some ejector designs, it is possible to increase the capacity by fitting a larger diameter nozzle and venturi. This gives an increased induced flow rate at a given supply pressure, at the expense of an increase in the required supply flow per ejector.

The supply pumps should be chosen to be able to supply the required total supply flow at the necessary pressure, taking into account friction losses in the system. Adequate standby pumping capacity should be provided. Supply pumps are typically high-speed single-stage or multistage centrifugal pumps; the pump and motor may be either horizontally or vertically coupled (Figure 11.5). Pumps are typically electrically powered, but units with diesel prime movers can be used in remote locations or for emergency projects. If the system consists of relatively few ejectors, one pump may be used to supply the whole system. However, large systems (more than

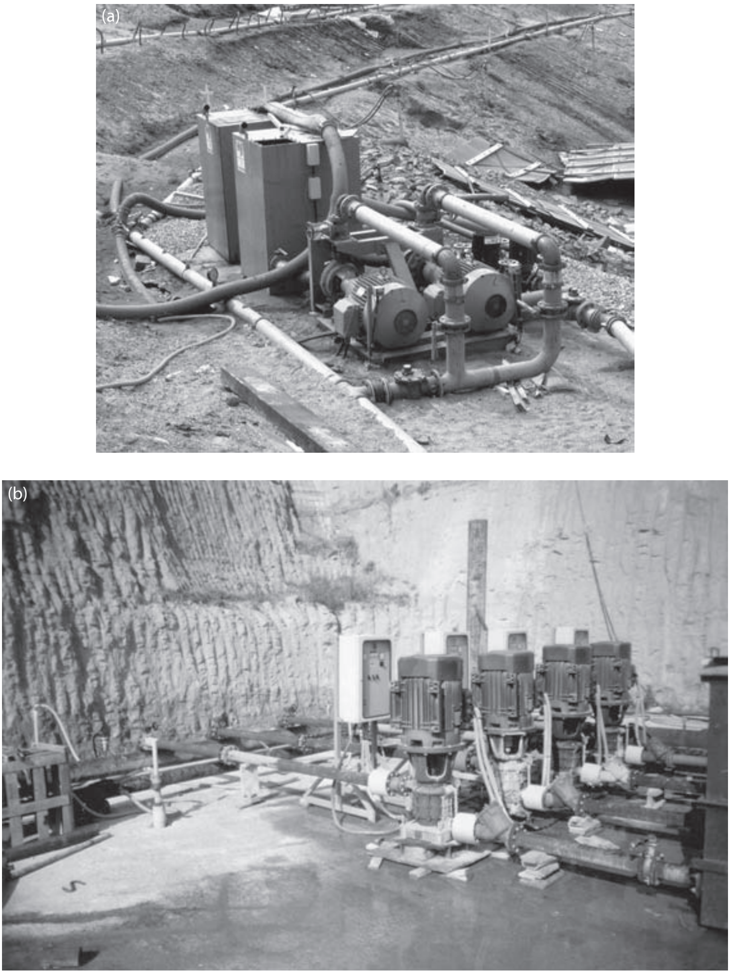


Figure 11.5 Ejector supply pumps. (a) Horizontally coupled pumps. (b) Vertically coupled pumps. (Courtesy of WJ Groundwater Limited, Bushey, U.K.)

Figure 11.6 Ejector pipework. An ejector headworks is shown fitted to the well. The supply and return header pipes can be seen either side of the ejector well, linked to the ejector by flexible hoses. (Courtesy of WJ Groundwater Limited, Bushey, U.K.)

15–25 ejectors) could be supplied by one large duty pump (Figure 11.5a) or a bank of several smaller duty pumps connected in parallel (Figure 11.5b). The use of several smaller pumps can allow a more flexible approach with additional pumping capacity being easy to add as needed and allows more economical use of standby pumps.

The supply pipework is normally made of steel with the pipes and joints rated to withstand the supply pressure. The return pipework does not need so large a pressure rating but may still be under considerable positive pressure, especially if the wells are drawing air into the system. Supply header pipes are typically 100 or 150 mm in diameter. Return header pipes are typically 150 mm in diameter, although bigger diameters may be appropriate if large numbers of ejectors are in use; air elimination valves may need to be incorporated in the return header to prevent air locking. Figure 11.6 shows the connection between the pipework and an individual ejector well.

The components of an ejector setup form a complex hydraulic system. For all but the smallest of systems, the design of the pumping systems should be carried out with care. The work of Miller (1988), Powrie and Preene (1994b), and Powers et al. (2007) is recommended as further readings to those faced with such a problem.

11.2.5 Operation of an ejector system and potential imperfections

The operation of an ejector system is relatively straightforward. In general, individual ejectors do not need adjusting or trimming, as might be required for wellpoints. The system should be very stable in operation, and the supply pressure (displayed on gauges at the supply pump) should hardly vary once the system is primed and running. Any changes in supply pressure may indicate a problem with the system and should be investigated.

Successful long-term operation of ejector systems relies on ensuring that the circulation water is not contaminated with suspended fine particles of soil or other detritus. Significant amounts of suspended solids will damage the supply pumps (which are generally intended for clean water only) and may build up in the ejector risers and bodies, restricting flow. However, the most serious effect of suspended solids is excessive wear of the nozzle and venturi in the ejector due to the abrasive action of the particles as these pass through the nozzle at high velocity. Even low levels of suspended solids can cause nozzle wear over weeks or months of pumping. As the nozzle wears, its opening enlarges, the supply pressure falls, and system performance deteriorates. Figure 11.7 shows examples of pristine, moderately worn, and severely worn nozzles.

Suspended solids may enter the circulation water in a number of ways:

1. The material may have been present when the system was assembled. If appropriate supervision and workmanship are not employed, it is not unknown for tanks and pipework to contain sand, dead leaves, and other extraneous material left over from previous use or storage. If these are not cleaned out prior to commissioning the system, the system will probably either clog up after a few minutes or suffer severe nozzle wear over the next few days; neither is particularly satisfactory. When installing a system, the pipes should be as clean as practicable. Prior to start-up, the system must be primed with *clean* water. The water must be run to waste to flush the system clean; flushing should only stop when the water runs clear. It is important to allow for this flushing out when estimating the volume of water needed for priming.
2. Silt or sand particles may be drawn from the wells into the system, either during initial stages of pumping (due to inadequate development) or continually (due to ineffective well filters). These problems

Figure 11.7 Examples of ejector nozzle wear. (a) New nozzle. (b) Moderately worn nozzle, after 6 months in slightly silty system. (c) Severely worn nozzle after 1 month in a moderately silty system. (Courtesy of W. Powrie.)

should be avoided by ensuring that appropriate well filters are in place and that development is not neglected.

3. Silt or sand may be drawn in from an individual well that may have a poorly installed filter or where the well screen has been fractured by ground movement. If this well is not identified and switched off, it will continually feed particles into the system. The circulating action means that the sand from one well can damage all the ejectors in a system.
4. Soil or debris may have inadvertently been added to the circulation tank as a result of construction operations. This has occurred, where spoil skips have been craned over the tank location, and small amounts of soil have fallen into the tank. This can be avoided by fitting a lid to the tank.
5. The growth of biofouling bacteria (see Section 16.9) can generate suspended solids in the form of iron-related compounds associated with the bacteria's life cycle. This is less straightforward to deal with, but one of the simplest solutions is to periodically dispose of the circulation water and flush out the system with clean water.

If significant nozzle wear does occur, once the cause of the problem has been identified and dealt with, the ejectors will need to be removed from the wells, and the worn nozzles and venturi should be replaced with pristine items. If there are many ejectors in the system or they are particularly deep, this can be quite an undertaking. Nozzle wear is definitely one case where prevention is better than cure.

Biofouling (item 5 above) may also cause problems by allowing material to build up in and around the ejectors, causing clogging rather than wear. In general, if the level of dissolved iron in the groundwater is more than a few milligrams per liter, the potential for clogging should be considered (see Section 16.9).

11.3 HORIZONTAL WELLPOINTS

The horizontal wellpoint method uses a horizontal flexible perforated pipe, pumped by a suction pump, to effect lowering of water levels. Typically, the perforated pipe (the horizontal wellpoint) is installed by a special land-drain trenching machine (Figure 11.8). One end of the pipe is unperforated and is brought to the surface and connected to a wellpoint suction pump. A horizontal wellpoint is very efficient hydraulically, because it has a very large screen area, and horizontal flow will be plane to the sides of the perforated pipe. This contrasts with flow local to vertical wellpoint systems, where flow lines converge radially to each wellpoint, and the screened area is limited by the short length of the wellpoint screens.

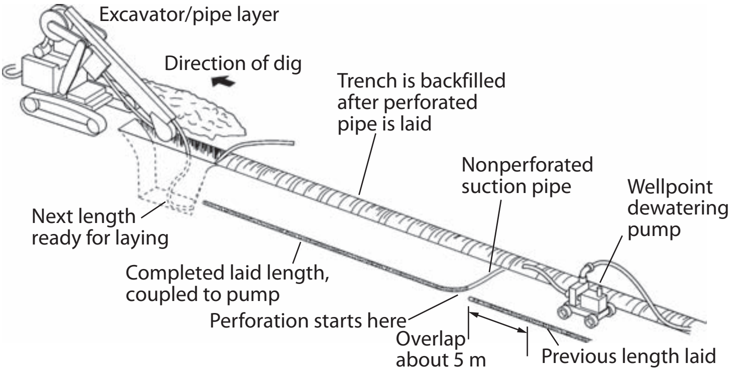


Figure 11.8 Horizontal wellpoint installation using a land-drain trenching machine. (From Preene, M. et al., Groundwater control—design and practice, Construction Industry Research and Information Association, *CIRIA Report C515*, London, 2000. Reproduced by kind permission by CIRIA: www.ciria.org.)

The main use of the method is for large-scale shallow cross-country pipelines as an alternative to single-stage wellpointing when rapid rates of installation and progression are required. The principal restriction on the use of the method is the local availability of the specialist trenching machines to lay the perforated pipe at adequate depth. Trenching machines capable of installing drains to 6–8 m depth are relatively commonly available in Holland and North America but are less readily available in the United Kingdom. Large trenching machines were used on British pipeline and motorway cutting projects in the 1960s and 1970s, but nowadays, most of the trenching machines in use in the United Kingdom are limited to installation depths of 3–4 m, although a small number of machines with deeper capability (6–7 m) are in use.

In addition to pipeline work, horizontal wellpoints are occasionally used instead of vertical wellpoints to form perimeter dewatering systems around large excavations for dry docks and the like (Anon 1976). If large drawdowns are required, multiple stages of horizontal wellpoints can be installed similar to vertical wellpoint systems. Horizontal wellpoints have also been used to provide groundwater abstraction systems for water use in shallow sand aquifers (Brassington and Preene 2003).

Although the horizontal method is best suited to the long straight runs associated with pipelines or the perimeter of large excavations, it can also be used to form a grid or herringbone pattern to dewater large areas to shallow depth. The method has also been used to consolidate areas of soft soils in conjunction with vacuum pumping systems (Anon 1998).

11.3.1 Merits of horizontal wellpoint systems

There are two principal practical advantages to the use of the horizontal wellpoint method. First, very rapid rates of installation can be achieved by specialist trenching machines (up to 1000 m/day in favorable conditions); this can be vital when trying to keep ahead of the installation of cross-country pipelines. Second, the absence of vertical wellpoints and surface header pipes alongside the trench allows unencumbered access for the pipe-laying operations. This has the additional benefit that there are fewer aboveground dewatering installations that might be damaged by the contractor's plant, with the associated risk of interruption of pumping.

Other practical advantages are listed as follows: (1) a supply of jetting water is not necessary for installation and (2) once the drainage pipe has been laid, installation and dismantling is simple and rapid, because only the pumps and discharge pipes are involved, without the need for header pipes.

From the cost point of view, although the horizontal drain cannot be recovered and is written off on the job, the cost of disposable vertical wellpoints and the hire cost of header pipes are saved. This can make the installation rate per meter of trench very cost-effective. However, overall costs should include for mobilization and demobilization costs, which can be high (in comparison to conventional wellpoint equipment) for large trenching machines and supporting equipment. This may be one of the reasons why this method tends not to be used on smaller contracts.

11.3.2 Installation techniques

Horizontal drains can be installed using conventional trench excavation techniques (e.g., by hydraulic excavator). However, these methods tend to be slow and may have problems in maintaining the stability of the trench while the drain is placed; such methods will only be cost-effective on relatively small projects or where rapid progression is not desired.

Horizontal wellpoint systems are more commonly installed using crawler-mounted trenching machines (Figure 11.9) equipped with a continuous digging chain, which typically cuts a vertical sided trench of 225-mm width as the machine tracks forward. Typical depths of installation are between 2 and 6 m, although deeper depths can be achieved by the very largest machines. A reel of flexible perforated drainage pipe feeds through the boom supporting the digging chain and is laid in the base of the trench. As the machine tracks forward, either the spoil is allowed to fall back into the trench or the trench is backfilled with filter media.

The perforated pipe used as the drain is typically unplasticized polyvinyl chloride (uPVC) or high-density polyethylene (HDPE) land drain of 80–100 mm diameter (although 150 mm pipe is sometimes used); the pipe is generally wrapped in a filter of geotextile mesh, coco matting, or equivalent. The pipe

Figure 11.9 Specialist trenching machine for installation of horizontal drains. (Courtesy of Hölscher Wasserbau GmbH, Haren, Germany.)

comes in continuous reels, perhaps 100 m in length. One end of the pipe is sealed with an end plug, and the other end is unperforated for the first 5–10 m. When the machine starts to cut the trench, the unperforated end is fed out first and is left protruding from ground level. The machine tracks away, cutting the trench and laying the drain almost simultaneously (Figure 11.10).

Figure 11.10 Trenching machine cutting trench and laying horizontal drains. (Courtesy of Hölscher Wasserbau GmbH, Haren, Germany.)

When the reel of drain runs out, the sealed end is left in the base of the trench. A new reel of drain is fitted to the machine, and the next section is laid in a similar way, with around a 5-m overlap between sections (Figure 11.8). Some trenching machines have the facility for the addition of filter media above the drain to improve vertical drainage in stratified soils. Filter media should be selected on the same basis as for vertical wellpoint systems (see Section 9.4.8).

When installing a horizontal drain for pipeline dewatering, the topsoil is normally stripped off prior to installation. This is a normal practice to allow the site to be reinstated at the end of the project but has the added advantage of allowing the trenching machine to track on the firmer subsoil. Some of the larger machines weigh up to 32 tonnes and can be difficult to operate on soft soils. It may be necessary to fit wider crawler tracks to reduce ground pressures. Many trenching machines can start trenching from ground level by rotating the digging boom into the ground while the digging chain is cutting. However, in many cases, a starter trench is dug by a conventional excavator, which allows the trenching machine to start cutting with the boom in the vertical position.

For pipeline works, the drain is typically installed along the centerline of the proposed pipeline, at a depth below the pipeline formation level (Figure 11.11). When pumped, this allows the drain to directly dewater the area beneath the proposed pipeline. When construction is complete, the horizontal drain is left in place and abandoned. Occasionally, it is necessary to grout up the drain at the end of the project to prevent any influence on long-term groundwater conditions and avoid the creation of an artificial horizontal pathway for groundwater flow (see Section 15.5).

The specialist trenching machines can be effective tools but are not without their problems. First, it can be difficult to detect unexpected ground conditions through which the drain has been laid. If coarse gravel, cobbles,

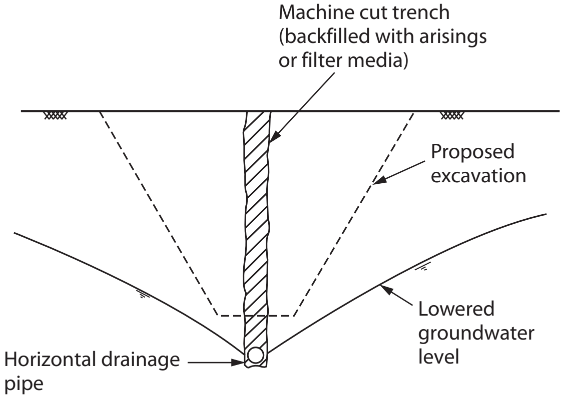


Figure 11.11 Installation of horizontal drains for pipeline trench.

or boulders are present, progress may be slowed, and wear to the trenching machine may be excessive; in extreme cases, there is a risk of the digging chain breaking. If layers of soft clay are present in conjunction with a high water table, the clay may "slurry up" and coat the perforated pipe, clogging it as it is laid. If such ground conditions are anticipated, judgment should be applied before committing to the horizontal wellpoint method.

11.3.3 Pumping equipment

Horizontal wellpoints are pumped by connecting a conventional wellpoint pump to the unperforated section of the drainage pipe where it emerges from the ground. As the horizontal system is pumped on the suction principle, it is subject to drawdown limitations for similar reasons to the vertical wellpoint system. In general, the maximum achievable drawdown will be limited to between 4.5 and 6 m depending on ground conditions and the type of pump used (see Chapter 13).

11.4 HORIZONTAL WELLS

Horizontal drilled wells are a technique used occasionally as part of groundwater control on civil engineering projects. This section describes the horizontal directional drilled (HDD) method, rather than subhorizontal drilled rock drains that are widely used to depressurize slopes in open pit mines (Leech and McGann 2008).

Horizontal wells are often considered for use where groundwater must be abstracted from beneath inaccessible areas or from areas where the disruption associated with surface drilling is undesirable. Applications have included installation of permanent dewatering systems (see Chapter 14) beneath existing built-up areas (Figure 11.12) and to extract contaminated groundwater or leachate (Cox and Powrie 2001) without the risks of cross-contamination associated with vertical drilling. HDD wells have also been used as part of dewatering during tunnel construction. HDD wells can act as preferential groundwater flow pathways to feed water in a controlled manner to the tunnel face or to a reception shaft ahead of the tunnel drive, from where the water is pumped away (Peter Cowsill Limited 2001). HDD wells have also been used as recharge wells to reinject water as part of artificial recharge schemes.

HDD techniques used for installation of dewatering wells have been developed from techniques originally developed to allow oil and gas pipelines to be drilled beneath obstacles such as roads and rivers. The methods are essentially based on rotary drilling, with a drilling fluid (mud) used for borehole support, cooling/lubrication of the drill bit, and transport of cuttings. The main difference in dewatering applications is that the mud

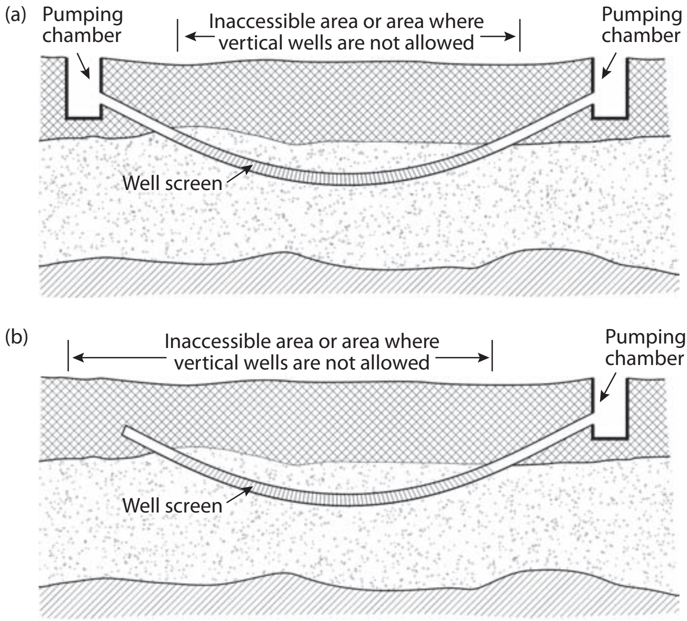


Figure 11.12 Horizontal well to pump from beneath urban area. (a) Double-entry completion. (b) Single-entry completion.

used to support the hole during drilling is normally based on biodegradable polymer, rather than bentonite, to reduce reduction of permeability associated with the drilling process. HDD installation lengths in excess of 500 m are possible.

The successful design and installation of HDD wells is challenging, and the unit cost of a typical HDD well will be high due to the length of the well and the complexity of the operation. It is essential that experienced designers and contractors are consulted when planning HDD well systems.

HDD wells are categorized as double- or single-entry completions based on the installation method.

Double-entry completions (Figure 11.12a). A pilot hole is drilled from a launch pit. The bore typically is angled down on entry, is steered to run approximately horizontally beneath the target zone, and then is steered upward to emerge at a reception pit. The pilot bit is then removed from the drill string and replaced with a larger reamer bit, which is then pulled back through the borehole to enlarge it to the required diameter. In some configurations, additional drill rods are added at the reception pit behind the reamer as it is pulled through. A series of reamers of progressively increasing size may be used to enlarge the hole to a final diameter. Double-entry completions are the most commonly used method, because better control of hole stability is possible compared to single-entry completions.

Single-entry completions (Figure 11.12b). This method (also known as a blind-ended hole) is used when there is no access for a reception pit and the well must be installed from one end only. Once the pilot hole is drilled, the bit is withdrawn, and a succession of reamers of progressively increasing size is used to enlarge the hole. Because reaming is carried out by pushing the bit (rather than pulling it as in double-entry completions), there is a risk that the enlarged hole will not follow the pilot bore. Well lengths that can be achieved with single-entry completions are significantly less than for double-entry methods.

Installation of well screen is much more difficult in HDD wells than for conventional vertical wells, primarily due to the long length of the wells and the fact that the wells are deliberately deviated in direction. This results in large tensile stresses on the screen during installation.

Installation of the string of well screen is normally achieved by being pulled through the hole by the drill rods (in single-completion wells, the well screen must be pushed in from the entry pit). Well screens are normally formed from HDPE, carbon steel, or stainless steel. The percentage open area of the screen is typically lower than for vertical wells to ensure high tensile and compressive strength of the screen. It is difficult to install a conventional granular filter pack around a HDD well screen; thus, it is common practice instead to use prepacked gravel filter, mesh, or geotextile filter screens (see Section 10.3.3) suitably protected to survive the screen being pulled through the well during installation.

Development of HDD wells after completion of drilling is also problematic, because the small open areas of the well screen and the types of filters used mean that much of the development energy is dissipated in the screen and filter system and never reaches the ground. Typically, water jetting and intermittent pumping and flushing with pressurized water are used to develop HDD wells.

11.5 PRESSURE RELIEF WELLS

When an excavation is made into a low-permeability layer above a confined aquifer, there is a risk that pore water pressures in the confined aquifer may cause the base of the excavation to become unstable. The base of the excavation may “heave,” because the weight of soil remaining beneath the excavation is insufficient to balance the uplift force from the aquifer pore water pressure (see Section 4.6). One way of avoiding this potential instability is to reduce pore water pressures in the aquifer by pumping from an array of deep wells (or, for shallow excavations, wellpoints).

Such pumped well systems are called "active" because pumps are used. The contrasting system of pressure relief (or bleed) wells offers an alternative method of reducing pore water pressures in confined aquifers. These systems are "passive"; this means that they are not directly pumped but merely provide preferential pathways for water from the aquifer to "bleed" away, driven by the existing groundwater heads.

A schematic view through a typical relief well system is shown in Figure 11.13. The wells are normally drilled prior to commencement of excavation or, at least, before the excavation has progressed below the piezometric level in the aquifer. As excavation continues, the wells will begin to overflow, relieving pore water pressures in the aquifer and ensuring stability. The water flowing from the relief wells is typically disposed of by sump pumping (see Chapter 8).

The discussion of relief wells described in this section is mainly restricted to wells of relatively large diameter (greater than 100 mm) formed by drilling or jetting and backfilled with sand or gravel. This type of well is distinct from the smaller diameter vertical drains (sometimes known as wick drains or prefabricated vertical drains) installed for soil consolidation purposes,

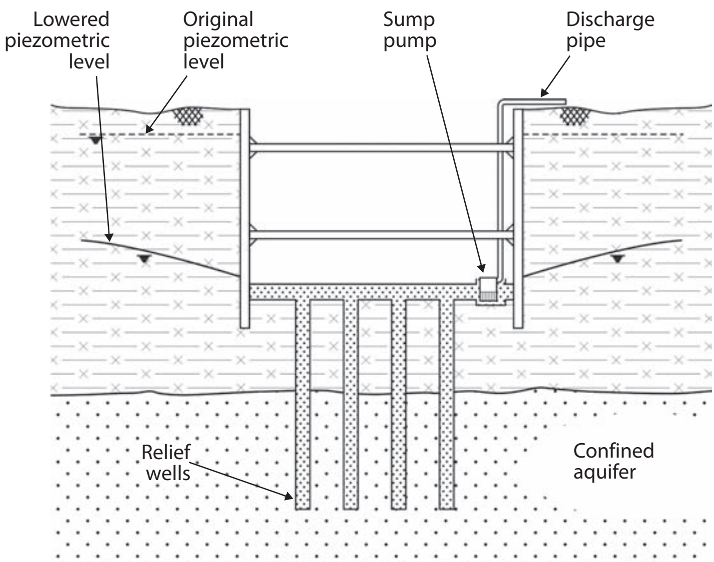


Figure 11.13 Relief well system. The relief wells overflow into a granular drainage blanket in the base of the excavation. The water is pumped away from a sump in the drainage blanket.

often formed using a mandrel to push plastic drainage wicks into soft clay and silt soils (Institution of Civil Engineers 1982).

11.5.1 Merits of pressure relief well systems

When used in appropriate conditions, the principal advantages of relief well systems are cost and simplicity. Typical relief wells consist of a simple gravel-filled borehole; because the wells do not need to accommodate pumps or well screens, they can be of modest diameter, reducing drilling and installation costs. The water flowing from the wells is removed by conventional sump pumps, which are more readily available and more robust in use than borehole electro-submersible pumps used to pump from deep wells.

Relief wells are best employed in shafts or deep cofferdams where the sides of the excavation are supported by a structural cutoff wall or other retaining structure and the stability of the excavation base is the primary concern. The method is most appropriate to use where the excavation base is in stiff clay or weak rock (such as chalk, soft sandstones, or fractured mudstones). Because the water from the relief wells overflows onto the excavation formation, there may be a risk of the water causing softening of exposed soils (especially in clays that have a permeable fabric). This can lead to difficult working conditions. It may be possible to avoid this problem by installing a granular drainage blanket and network of drains to direct water to the sumps and prevent ponding in the excavation.

A key question when considering a relief well system is: How many wells of a given diameter will be required? An initial stage is obviously to estimate the rate at which groundwater must be removed by the wells to achieve lowering to formation level; typically, this is estimated by treating the excavation as an equivalent well (see Section 7.6). Theory suggests that the capacity of a gravel-filled vertical drain can be estimated by the direct application of Darcy's law (see Section 3.3)

$$Q = kiA \tag{11.1}$$

where Q is the vertical flow rate along a relief well (in cubic meters per second), k is the permeability of the gravel backfill (in meters per second), A is the cross-sectional area of the well (in square meters), and i is the vertical hydraulic gradient along the well.

The vertical hydraulic gradient in the relief well is often taken as unity. Table 11.1 presents the theoretical maximum capacity of relief wells based on Equation 11.1, assuming fully saturated conditions and unit hydraulic gradient.

Although many engineers will find these values surprisingly low, field experience suggests that these values are theoretical maximums, rarely

Table 11.1 Maximum theoretical capacity of sand- or gravel-filled relief wells

Permeability of filter backfill (m/s)	*100-mm borehole (L/min)*	*150-mm borehole (L/min)*	*300-mm borehole (L/min)*
1×10^{-4}	0.05	0.11	0.42
5×10^{-4}	0.24	0.53	2.1
1×10^{-3}	0.47	1.1	4.2
5×10^{-3}	2.4	5.3	21
1×10^{-2}	4.7	11	106
5×10^{-2}	24	53	530

achieved in practice. In fact, the actual capacity may be significantly less than the theoretical capacity for a variety of reasons, including smearing or clogging of the borehole wall during drilling or segregation of the filter material during placement.

Although the number of relief wells required must consider the well capacity, some thought must also be given to the spacing between wells. If small flows are predicted, only a few wells may be necessary to deal with the volume of water. However, it may be prudent to install additional relief wells to ensure that the distance between wells is not excessive (e.g., greater than 5–10 m). If the wells are widely spaced and the ground conditions may be variable (especially if the wells are installed into fissured rock), there is a danger that the wells may not adequately intercept sufficient permeable zones or fissures. This could lead to unrelieved pressures remaining, with the possibility of local base heave in the areas between the wells.

11.5.2 Installation techniques

Relief wells are typically drilled by similar methods to deep wells, e.g., cable percussion boring, rotary drilling, jetting, or auger boring (see Chapter 10). The borehole is drilled to full depth, any drilling fluids used are flushed clear, and then, the filter media (sand or gravel) are added to backfill the bore up to the required level. The diameter of drilling is typically between 100 and 450 mm.

The filter media may be placed in the well via a tremie pipe or may simply be poured in from ground level. This latter approach is acceptable, provided that the filter medium has a very uniform grading so that there is little risk of segregation of the filter particles as they settle to the bottom of the bore. In practice, many relief wells installed in soft rocks, where the performance of the filter is not critical, are filled with uniform coarse gravel. Ideally, to permit maximum transmission of water, the gravel should be of the rounded pea shingle type of 10–20 mm nominal size. However, on remote sites, it may be necessary to use locally available material (which may consist of angular crushed aggregates) and accept some reduction in well efficiency. In certain cases, where the long-term performance of the relief wells is critical, the gravel used may need to be designed to match soil conditions similar to deep wells (see Section 10.3). If no well screen is installed, it is not normally possible to develop the relief well.

11.5.3 Relief wells—Is a well casing and screen needed?

Previous sections have mainly discussed gravel-filled relief wells, but there are cases when it is appropriate to install casings and screens (surrounded by a gravel pack) in relief wells. Although they introduce additional cost and complexity, relief wells with screens can be used in the following cases:

1. If the confined aquifer is of high permeability, a screened well will have a greater vertical flow capacity than a purely gravel-filled well. The use of screened wells may reduce the number of wells required.
2. If a pumping test is needed to confirm aquifer permeability and flow rate, the casing and screen will allow the test to be carried out using a submersible pump. This may be appropriate for large excavations where a widely spaced grid of relief wells is installed based on an initial assessment of permeability. Pumping tests are then carried out on some of the wells to estimate the actual permeability and determine whether additional relief wells are needed to fill in the gaps between the original wells.
3. If, during critical stages of construction (e.g., during casting of concrete structures), overflowing water would be an inconvenience, it may be possible to install submersible pumps in the wells and temporarily lower water levels below formation level. This would allow the critical works to be completed in more workable conditions.

Relief wells are sometimes also used as part of the permanent works to provide long-term pressure relief for deep structures (e.g., basements or deep railway cuttings) constructed above confined aquifers (see Section 14.3). Permanent relief wells are often installed with screen and casings to allow the wells to be cleaned out and redeveloped if their performance deteriorates after several years of service.

11.6 COLLECTOR WELLS

A collector well consists of a vertical shaft typically 3–6 m in diameter, sunk as concrete-lined caisson, from which laterals (horizontal or subhorizontal screened wells, typically 200–250 mm in diameter) are jacked or drilled radially outward (Figure 11.14). Collector wells are sometimes known as Ranney wells after a proprietary system (first developed by Leo Ranney in 1933) used to form this type of well (Drinkwater 1967). Ranney's concept was that, by placing well screens horizontally into a permeable formation, a much greater length of screen would be productive and, therefore, a collector well will have a much greater yield than a conventional vertical well.

A collector well is pumped by a submersible or lineshaft pump located in the shaft, which lowers the water level in the well. This creates a pressure gradient along the laterals, causing them to flow freely into the main shaft. This method is particularly suited to large-capacity permanent water supply installations in shallow sand and gravel aquifers. Water supply installations are described in the work of Hunt (2002) and Hunt et al. (2002).

Collector wells are rarely used for groundwater control operations. In general, the cost of constructing the central shaft and forming the laterals is likely to be prohibitive for temporary work applications. Nevertheless, the method has occasionally been used, for example, to aid dewatering of tunnel crossings beneath roads and railways where access to install conventional wells was restricted (Harding 1947). In the early 2000s, four collector wells were used to form a long-term dewatering system beneath a tunnel in Glasgow in Scotland (Schünmann 2005). This project is described in more detail in Section 14.7.

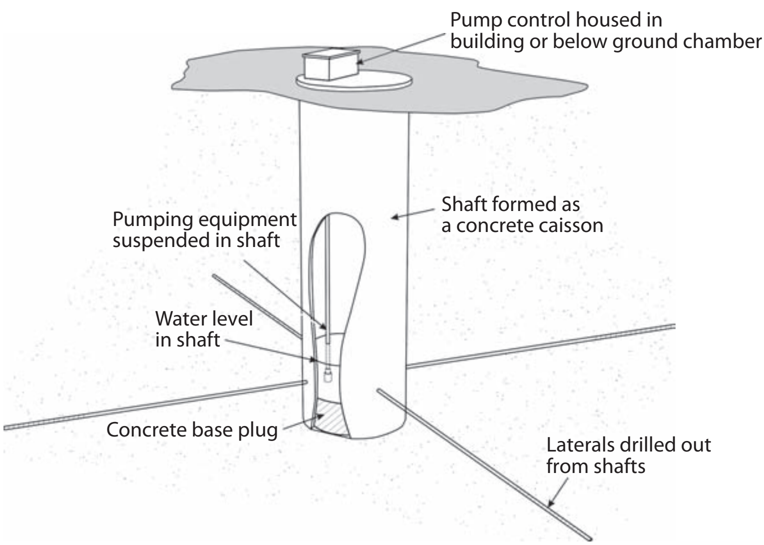


Figure 11.14 Collector well. Pumping from the shaft of the collector well creates a hydraulic gradient along the lateral wells, which draws water into the shaft.

11.7 SIPHON DRAINS

The technique of siphon drains was developed in the 1980s to allow water to be pumped by gravity (without the need for external energy input) via permanently primed siphon pipes (Mrvik et al. 2010). This approach has found useful applications in landslide drainage and stabilization schemes where long-term pumping at low flow rates is required and the sloping terrain allows the siphon system to be arranged at appropriate levels (Bomont et al. 2005).

Siphon drain systems are used to pump from wells located in the zone that is to be drained or depressurized. Wells are typically at approximately 5-m spacing. The suction end of the siphon pipe is installed in the well in a container so that the end of the pipe remains submerged below water even if the water level in the well falls below the level of the siphon pipe inlet. The top of the container is set at the target drawdown level in the well (known as the "reference level"). The siphon pipe is connected at ground level to an accumulator located downslope at the reference level (Figure 11.15). The siphon pipe is usually installed in a duct and the accumulators in an outlet manhole all at a suitable depth below ground level for frost protection. The purpose of the accumulator is to maintain the prime in the siphon.

Once primed, when the water level in the well is above the "reference level," water will flow through the siphon (to be discharged at the outlet). Flow will continue until the water level in the well falls to the "reference level" when flow will slow and stop. At the top of the siphon tube, the water pressure will be low, and gas may come out of solution, creating bubbles. These gas bubbles are flushed out when the siphon is flowing, but if the flow

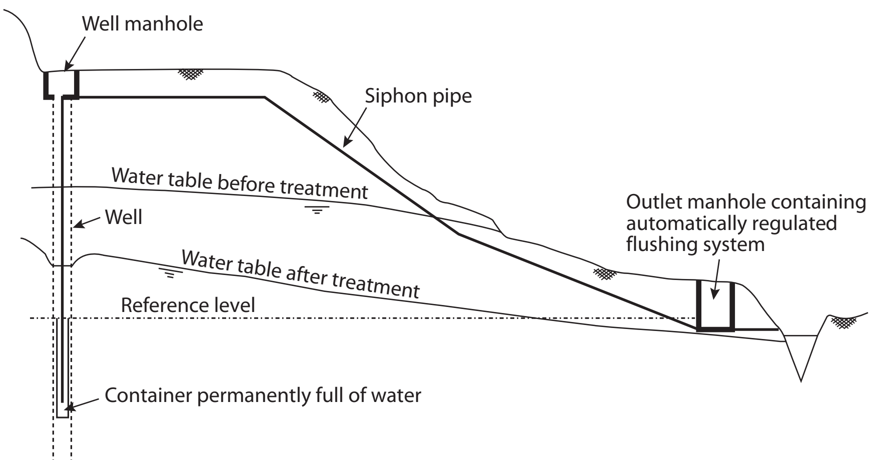


Figure 11.15 Siphon drain system.

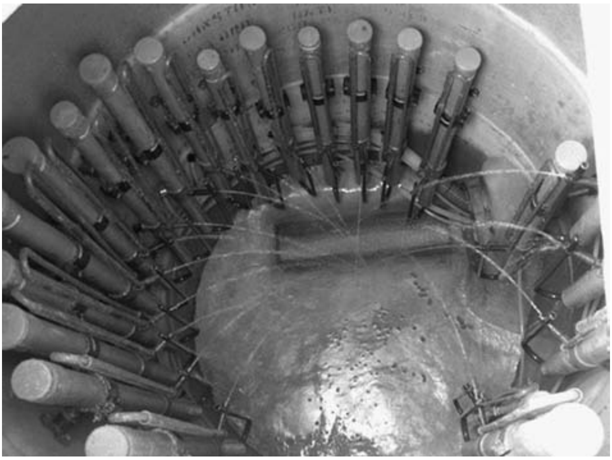


Figure 11.16 Accumulator flushing system for siphon drains. The photograph shows a view looking down into the base of an outlet manhole for a system of several siphon drains. The accumulators are flushing the system to remove gas that may cause prime to be lost. Water is being discharged into the base of the outlet manhole. (Courtesy of TPGEO, Fontaines, France.)

slows and stops, the gas bubbles may build up, potentially leading to the loss of prime in the siphon. When the water level in the well rises, the accumulator automatically reprimes the siphon, flushing out any accumulated gas bubbles in the line, and restarts the flow. Typically, the siphon pipes from several wells would be plumbed into a single outlet manhole. Figure 11.16 shows the accumulator flushing system from several wells in operation in an outlet manhole.

The siphon drain technique is suited to soils of relatively low-permeability (less than 1×10^{-5} m/s). Depending on the size of the equipment, flow rates of up to 15 L/min (0.25 L/s) are possible. Drawdowns of up to 8.5 m can be achieved below the top of the siphon pipe.

11.8 ELECTRO-OSMOSIS

Electro-osmosis is suitable for use in very low-permeability soils such as silts or clays with no permeable fissures or fabric, where groundwater movement under the influence of pumping would be excessively slow. Electro-osmosis causes groundwater movements in such soils using electrical potential gradients rather than hydraulic gradients. A direct current is passed through the soil between an array of anodes and cathodes installed in the ground. The potential gradient causes positively charged ions and pore water around the soil particles to migrate from the anode to the cathode, where the small volumes of water generated can be pumped away by wellpoints or ejectors (Figure 11.17). The method can reduce the moisture content of the soil, thereby increasing its strength. In many ways, electro-osmosis is not so much a groundwater control method as a ground improvement technique. One of the drawbacks of the method is that it is a decelerating process, becoming slower as the moisture content decreases.

Electro-osmosis is a very specialized technique and is used rarely, mainly when very soft clays or silts are required to be increased in strength. Casagrande (1952) describes the development of the method. Some relatively recent applications are given by Casagrande et al. (1981) and Doran et al. (1995). In some applications, electro-osmosis is used in conjunction with electrochemical stabilization (Bell and Cashman 1986), when chemical stabilizers are added at the anodes to permanently increase the strength of the soil.

In application, the electrode arrangements are straightforward, typically being installed in lines, with a spacing of 3–5 m between electrodes. Anodes and cathodes are placed in the same line, in an alternating anode–cathode–anode sequence (Figure 11.17). Water is to be pumped from the cathodes; thus, they can be formed from steel wellpoints or steel well liners. If it is desired to use plastic well liners at the cathodes, a metal bar or pipe (to form the electrode) will need to be installed in the sand filter around the well (Figure 11.18a). The anodes are essentially metal stakes (Figure 11.18b); gas pipe, steel reinforcing bar, old railway lines, or scrap sheet piles can be used.

Applied voltages are generally in the range of 30–100 V. Effectiveness can be improved if the potential gradient can be in the same direction as the hydraulic gradient. Casagrande (1952) states that the potential gradient should not exceed 50 V/m to avoid excessive energy losses due to heating of

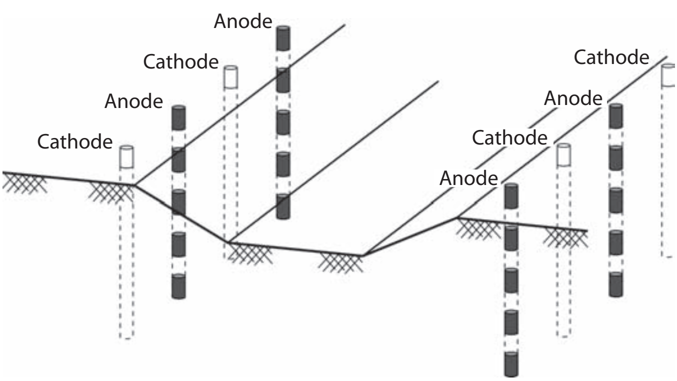


Figure 11.17 Electro-osmosis system. A direct current is applied to the electrodes. Water is drawn to the cathodes. The anodes can be simple metal stakes, but the cathodes are formed as wells and are pumped by wellpoints or ejectors.

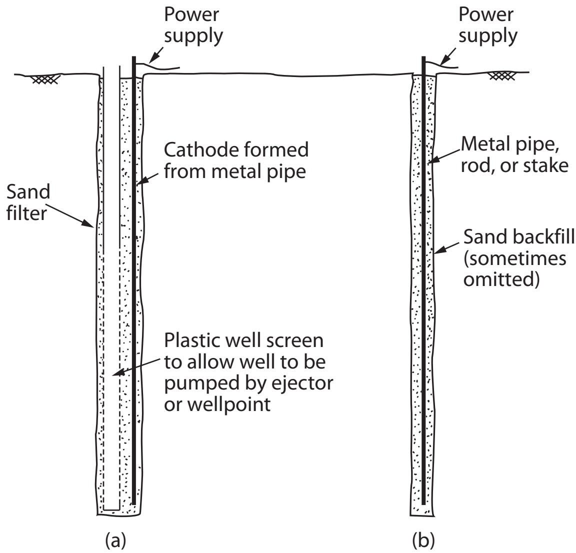


Figure 11.18 Typical electrode details. (a) Cathode well. (b) Anode.

the ground. However, it might be advantageous to operate at 100–200 V/m during the first few hours to give a faster build-up of groundwater flow. Reduction in power consumption may be possible if the system can be operated on an intermittent basis.

11.9 ARTIFICIAL RECHARGE SYSTEMS

Artificial recharge is the process of injecting (or recharging) water into the ground in a controlled way, typically by means of special recharge wells or trenches (Figure 11.19). Artificial recharge systems are not straightforward in planning or operation and are carried out on only a small minority of groundwater lowering projects. One of the most significant difficulties is the management and mitigation of clogging of recharge wells.

For groundwater lowering applications, water from various sources has been used for artificial recharge:

1. By far, the most common case is where the recharged water is the discharge from the dewatering abstraction system, injected into the ground to control or reduce drawdowns around the main excavation area, or simply as a means of disposing of the discharge water.
2. Less commonly, mains water from the municipal supply is used for recharge. This is normally done to reduce the risk of clogging of

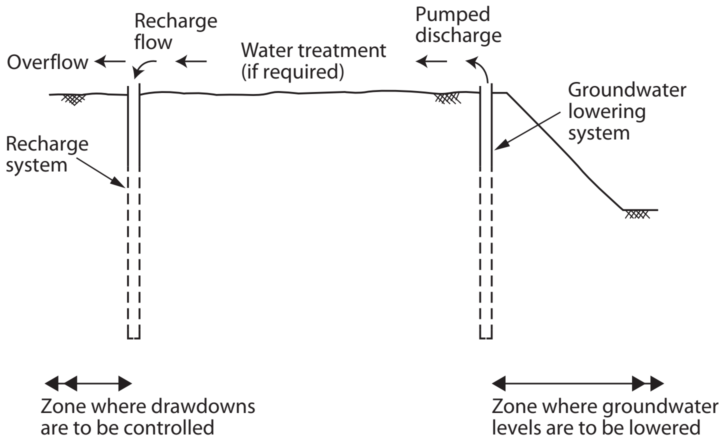


Figure 11.19 Artificial recharge to control drawdowns around a groundwater lowering system.

recharge wells when the quality of the dewatering discharge water indicates the likelihood of severe clogging problems. The downside of this approach is the high cost of the mains water.

3. On rare occasions, in coastal areas where saline intrusion of aquifers has occurred (see Section 15.4.11), it may be environmentally acceptable to recharge seawater into the ground. Bock and Markussen (2007) describe the artificial recharge system for the Copenhagen Metro Project, where in the harbor area where groundwater was already saline, water from the harbor was used for recharge purposes. It was assessed that the low dissolved iron content of the harbor water, relative to the pumped groundwater, would reduce the severity of clogging of recharge wells.

11.9.1 Applications of artificial recharge systems

When used in conjunction with groundwater lowering systems, artificial recharge can be used as a mitigation measure to control or reduce drawdowns away from the main area of groundwater lowering in order to minimize the potential environmental impacts described in Chapter 15.

Artificial recharge is also occasionally used as a means of disposing of some or all of the discharge water if other disposal routes are not practicable. If this option is being considered, it must be noted that, in many countries, including the United Kingdom, formal permission (see Section 17.4) must be obtained from the regulatory authorities to allow artificial recharge; there is no automatic right to recharge groundwater back into the ground. Similar regulations exist in several countries.

Recharge wells or trenches must be located with care. If the system is intended to reduce drawdowns at specific locations, then the recharge points will generally be between the groundwater lowering system and the areas at risk. The recharge locations may be quite close to the pumping system, and much of the recharge water may recirculate back to the abstraction wells, leading to an increase in the pumping and recharge rate. Unless the soil stratification can be used to reduce the connection between the pumping and recharge system, a physical cutoff barrier could be used to minimize recirculation of water through the ground. If the water is being recharged to prevent derogation of water supplies, the recharge should be carried out further away to avoid excessive recirculation.

It is worth noting that artificial recharge is being increasingly applied in the water supply field. Artificial recharge in that context involves injecting water (often treated to potable standards) into the aquifer at a location where there is a supply at ground level (e.g., from a river). This injection effectively provides more water to the aquifer that can be abstracted from the aquifer downgradient, where it is to be used. A variant on artificial recharge, used increasingly in the United States and Europe, is aquifer storage recovery (ASR). ASR involves the storage of potable water in an aquifer at one location. This is done by injecting water at times when supply is available and, later, when it is needed to be put back into supply, recovering the water from the same wells. This interesting application of groundwater technology is described in detail in the work of Pyne (1994).

11.9.2 What is the aim of artificial recharge?

If an artificial recharge system is being planned, it is vital that the aim of the system is clear at the start. It is sometimes thought that an artificial recharge system should prevent any drawdown or lowering of groundwater levels around a site. However, Powers (1985) suggests that the aim of a system should not be to maintain groundwater levels per se but to prevent side effects (such as settlement or derogation of water supplies) from reaching unacceptable levels.

Groundwater levels vary naturally in response to seasonal recharge and under the influence of pumping. Other natural effects include tidal groundwater responses in coastal areas. A specification for an artificial recharge system that required “zero drawdown” would not be practicable. It may be more appropriate to set a target such that groundwater levels in selected observation piezometers should not fall below defined levels (which are likely to be different in different parts of the site).

Assuming that groundwater levels around a site vary naturally with time, and that this variation is not causing detrimental side effects, the lowest acceptable groundwater level in a monitoring well is often set as the seasonal minimum. Occasionally, if the groundwater lowering system is intended to generate very large drawdowns in relatively stiff aquifers, the lowest acceptable groundwater level is set rather lower, at a few meters below the seasonal minimum.

It is sometimes also useful to specify maximum groundwater levels in selected piezometers. This reduces the temptation for overzealous recharging, raising the groundwater levels above the seasonal maximum, which could lead to problems with flooded basements and the like. By defining an allowable minimum and maximum groundwater level in each observation well (with at least 0.5 m between the maximum and minimum levels), the system operator has a little leeway within which to adjust the system.

11.9.3 Recharge trenches

A simple method of recharge for shallow applications is the use of recharge trenches, sometimes known as infiltration trenches (Figure 11.20a). These are excavated from the surface, and the dewatering discharge is fed into them from where the water infiltrates into the ground.

Recharge trenches have been employed to good effect when pumping from mineral workings (Cliff and Smart 1998) but are less effective in construction situations and are rarely used. Clogging of the trenches (by the growth and decay of vegetation or the build-up of sediment), reducing infiltration rates, is often a problem. Trenches typically require periodic cleaning out by excavators. It is difficult to control and measure the volumes entering the trenches, making this method less easy to adjust than a system of recharge wells. An overflow channel is normally required to prevent overtopping of the trenches.

The recharge trench method is best suited to unconfined aquifers with water tables near ground level, allowing shallow trenches to be used. The method is less practicable for recharging water where the water table is relatively deep or if the aquifer is confined by overlying clay layers.

An interesting variation on the recharge trench is described by Ervin and Morgan (2001), which they call a “hydraulic wall.” Their requirement was to control the settlement of a compressible silt layer by maintaining pore water pressures behind a groundwater cutoff wall. The silt was of low-permeability; thus, conventional recharge wells would have been of little use, because the flow capacity of each well would have been very small. The approach adopted was to construct a geotextile-lined gravel-filled trench, through the base of which prefabricated vertical drains (wick drains) were installed into the silt (Figure 11.20b). A header tank was used to keep the trench charged with water; thus, the vertical drains continuously fed water to the silt, counteracting any minor leakage through the cutoff wall and maintaining external pore water pressures.

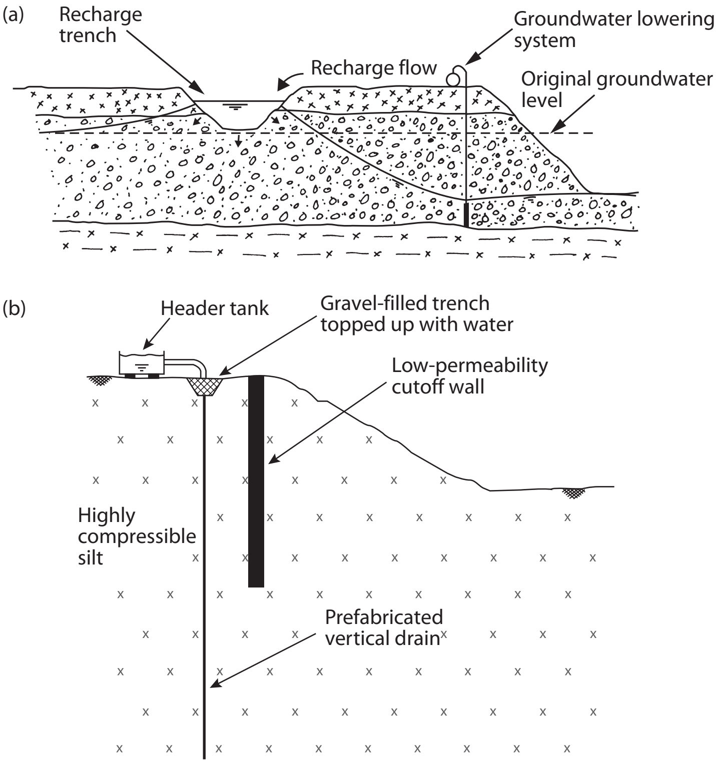


Figure 11.20 Recharge trench. (a) Recharge trench in unconfined aquifer. (b) Recharge trench used in combination with prefabricated vertical drains. (Based on Ervin, M.C., and Morgan, J.R., *Canadian Geotechnical Journal*, 38, 732–740, 2001.)

11.9.4 Recharge wells

Recharge wells (sometimes known as reinjection wells) are generally superior to recharge trenches, because the wells can be designed to inject water into specific aquifers beneath a site; this can allow the stratification of soils at the site to be used to advantage in controlling drawdowns. Recharge wells also allow better control and monitoring of injection heads and flow rates than do trench systems. A recharge well operates similar to a pumping well, except that the direction of flow is reversed. A recharge well should allow water to flow into the aquifer with as little restriction as possible. However, recharge of water into the ground is more difficult than pumping. A pumping well is effectively self-cleaning, because any loose particles or debris will be removed from the well by the flow. In contrast, a recharge well is effectively self-clogging: even if the water being recharged is of high quality, any suspended particles or gas bubbles will be trapped in the well (or the aquifer immediately outside), leading to clogging and loss of efficiency.

In broader terms, a recharge well should be designed, installed, and developed in the same way as a pumping well (see Chapter 10). The two key differences are

1. To maximize hydraulic efficiency, the well filter media and slot size (see Section 10.3) should be as coarse as possible while still allowing the well to be pumped without continuous removal of fines during redevelopment.
2. Because a recharge well does not generally have to accommodate a submersible pump, the well casing and screen can be of smaller diameter than a pumping well. However, the well casing and screen must be large enough to allow the well to be redeveloped if necessary.

Figure 11.21 shows a typical recharge well. Key features are

1. Well casing and screen, surrounded by filter media, with a grout or concrete seal around the well casing to prevent water short-circuiting up the filter pack to ground level. If the top of the well casing is also

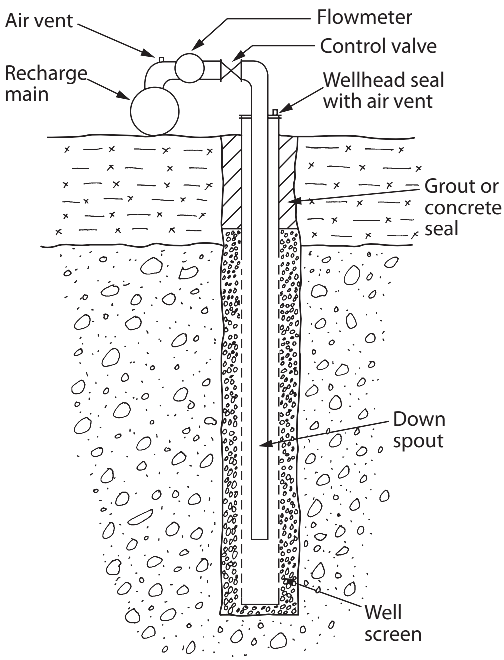


Figure 11.21 Recharge well.

sealed around the recharge pipework, this can allow recharge flows to be fed by header tanks raised above ground level to increase the injection rate in each well.
2. A downspout to prevent the recharge water from cascading into the well and becoming aerated. Aeration of the water may promote biofouling and other clogging processes and should be avoided as much as possible.
3. Air vents at the top of the well and pipework to purge air from the system when recharging commences, and to prevent air locks in the system.
4. Control valve and flowmeter to allow monitoring and adjustment of flow to the well.

If water is to be recharged into a shallow aquifer, a system of recharge wellpoints may be considered. The recharge wellpoints are installed at close spacings using similar methods to conventional wellpointing (see Chapter 9).

Almost all recharge wells suffer from clogging to some degree, and it is vital that the design used allows for appropriate redevelopment. This is discussed further in the following sections.

11.9.5 Water quality problems and clogging

Experience has shown that it is much harder to artificially recharge water into the ground than it is to abstract it. There are various rules of thumb stating that, to recharge water back into the aquifer from which it came, two or three recharge wells will be needed for every abstraction well. Many of the practical difficulties with artificial recharge arise from water quality problems leading to clogging. The designer and operator are being unrealistic if they do not expect recharge wells or trenches to clog (to a lesser or greater degree) in operation.

It is worthwhile to consider the mechanisms that can lead to the clogging of recharge wells (similar processes affect recharge trenches). Pyne (1994) has outlined five clogging processes:

1. *Entrained air and gas binding*. Bubbles of vapor (such as air or methane) present in the recharge water build-up in the well, inhibiting flow of water through the filter pack into the aquifer. The gas bubbles may result from the release of dissolved gases from the recharge water; from air drawn into the recharge pipework where changes in flow generate negative pressures or, if water is allowed to cascade into the recharge well, from air entrained into the water. Careful sealing of pipework joints and minimizing aeration of the recharge water can help avoid this problem. Degassing equipment can be installed onto the pipework to bleed off any gas before it reaches the recharge wells.
2. *Deposition of suspended solids from recharge water*. Particles (colloidal or silt- and sand-sized soil particles, biofouling detritus, algal

matter, loose rust, or scale from pipework) carried with the recharge water will build up in the well and filter pack, blocking the flow. Control of this problem requires effective design and development of the abstraction wells to minimize suspended solids in the water; the discharge from sump pumping is rarely suitable for recharge. For low-flow-rate systems, the use of sand filter systems to clean up the water might be considered.

3. *Biological growth*. Bacterial action in the recharge wells can result in clogging of the wells themselves. In addition, biofouling of the abstraction wells and pipework can release colloidal detritus (the result of the bacteria life cycle) into the water. The flow of water will carry these particles inexorably into the recharge wells, leading to further clogging. The severity of any clogging will depend on several factors (see Section 16.9). Because some of the problem bacteria are aerobic, minimizing any aeration of the recharge water is advisable. Periodic dosing of the wells with a dilute chlorine solution (or other disinfecting agents) has been used to inhibit bacterial growth.
4. *Geochemical reactions*. The recharge water can react with the natural groundwater or with the aquifer material. Such reactions are most likely if the recharge water is not groundwater from the same aquifer (e.g., if mains water is used for recharge or water from one aquifer is recharged back to another aquifer) or if the pumped groundwater is allowed to change chemically prior to reinjection. Typical reactions include the deposition of calcium carbonate or iron/manganese oxide hydrates. The potential significance of these reactions can only be assessed following study of the aquifer and groundwater chemistry but are generally reduced in severity if water pressure and temperature changes during recharge are minimized. Chemical dosing of the recharge water might be considered to inhibit particular reactions.
5. *Particle rearrangement in the aquifer*. Although not usually a significant effect, the permeability around the well may reduce due to loose particles around the well being rearranged by the flow of recharge water out of the well. This effect can be minimized by effective development on completion and periodic redevelopment.

These clogging effects should be considered when designing and operating an artificial recharge system. After all, a recharge system is not going to achieve its aims if it is unable to continue to inject water into the ground.

11.9.6 Operation of recharge systems

An artificial recharge scheme broadly consists of an abstraction system, a recharge system, and a transfer system (between the abstraction and recharge points). The abstraction system should be straightforward in operation and maintenance (see Chapter 16), but the transfer and recharge systems may be more problematic. The crux of the issue is to manage the recharge water quality to limit the clogging of recharge wells to acceptable levels that do not prevent the system from achieving its targets.

As previously stated, most artificial recharge systems will suffer from clogging. There are two approaches for dealing with this, often used in combination:

1. Prevention (or reduction) of clogging—principally by treatment of the discharge water to remove suspended solids and retard clogging processes such as bacterial growth.
2. Mitigation of clogging—accepting that clogging will occur, and then planning for it. This may involve providing spare recharge wells, to be used when efficiency is impaired, and implementing a program of regular redevelopment of clogged wells.

Measures of preventing clogging must focus on ensuring that the recharge water is of high quality and does not promote clogging processes. Possible measures include

1. Removal of suspended solids by filtration by sand filters or bag or cartridge filters. Filters will require periodic cleaning, backwashing, or replacement as solids collect in them.
2. Chemical dosing to precipitate out problematic carbonates or iron and manganese compounds.
3. Intermittent or continuous chlorine dosing of the recharge water to reduce bacterial action. It is also a relatively common practice to disinfect recharge wells with chlorine or other biocides on the first installation and following later rehabilitation.
4. Prevention of aeration of water to reduce the risk of gas binding.
5. The use of mains water if the pumped groundwater cannot be rendered suitable.
6. Careful management of the system on start-up to avoid slugs of sediment-laden water entering the wells (the initial surge of water may pick up sediment deposited in the pipework and should be directed to waste, not to the recharge wells).

Measures of mitigating clogging include

1. Designing recharge well screens to have ample open area to accept flow. Sterrett (2008) recommends that the average screen entrance velocity (recharge flow divided by total area of screen apertures) is less than 0.015 m/s, approximately half the maximum velocity recommended for abstraction wells.

2. Redeveloping the recharge wells on a regular basis to rehabilitate the wells (see Section 16.9). This is most commonly done by airlift development, sometimes in combination with chemical treatment, which dislodges any sediment, loose particles, precipitates, and bacterial residues. Depending on the severity of clogging, wells in a system may be redeveloped on a rolling program, with an individual well being redeveloped every few weeks or months. Even after redevelopment, wells may not return to their preclogging performance; if the system operates for very long periods, additional recharge wells may need to be installed after a few years to prevent the overall system performance from dropping to unacceptable levels.
3. Providing additional recharge wells over and above the minimum number of recharge wells necessary to accept the recharge flow. This way, the flow can still be accepted if some of the wells are badly clogged, allowing more time for redevelopment.

Monitoring of artificial recharge systems follows good practice for abstraction systems (see Chapter 16). For recharge wells, the water level below ground level (by dipping) or pressure head above ground level (by pressure gauge) should be monitored. Ideally, flowmeters should be installed on each well to record recharge flow rates, but some designs may be affected by clogging. At the very least, the overall recharge rate should be monitored, perhaps by V-notch weir.

11.9.7 Case history: Recharge to control settlement—Bank Misr, Cairo

An interesting application of recharge was carried out at the center of Cairo, Egypt, as described by Troughton (1987) and Cashman (1987). A two-story basement was to be constructed for the new Bank Misr headquarters immediately adjacent to the existing bank building. The basement was to be formed inside a sheet-piled cofferdam to a depth of 7.5 m below ground level and approximately 4.5 m below original groundwater level.

Although the sides of the excavation were supported by sheet piles, and a stratum of fissured clay was expected at final formation level, the presence of permeable sand layers below the excavation required reduction of pore water pressures at depth to prevent base heave. In planning the groundwater lowering operations, two issues were of concern:

1. To control drawdowns beneath adjacent structures (which were founded on compressible alluvial soils) to prevent damaging settlements.
2. To dispose of the discharge water, which could not be accepted by the local sewer network, which was heavily overloaded.

A system of recharge wells, used in combination with wellpoints pumping from within the excavation, was used to solve the problem. Without such a system, it is unlikely that the Cairo authorities would have allowed the work to proceed.

The sequence of strata revealed in the site investigation is shown in Figure 11.22. Falling head tests in the silty sand below formation level gave results of the order of 10^{-5} m/s, but pumping tests at the site indicated that the permeability was more probably of the order of 10^{-4} m/s, consistent with Hazen's rule applied to particle size tests on the sand. Due to the complexity of the interaction between pumping and recharge, a finite-element numerical groundwater model was set up for the site. Based on the soil fabric in the alluvial soils, the horizontal permeability was assumed to be significantly greater than the vertical permeability.

The model was used to predict the pore water pressure reduction needed to safeguard the base of the excavation, and to limit the drawdown beneath adjacent structures to less than 1 m. Calculations indicated that a 1-m drawdown would result in a 12-mm settlement of the fill and alluvial soils, with differential movements within acceptable limits. Figure 11.23 shows a schematic plan view of the system, with an abstraction wellpoint system inside the cofferdam, and recharge wells between the excavation and critical buildings.

Abstraction wellpoints were installed to pump from strata at two levels at 13.5 and 21.5 m below ground level. The recharge wells were installed in

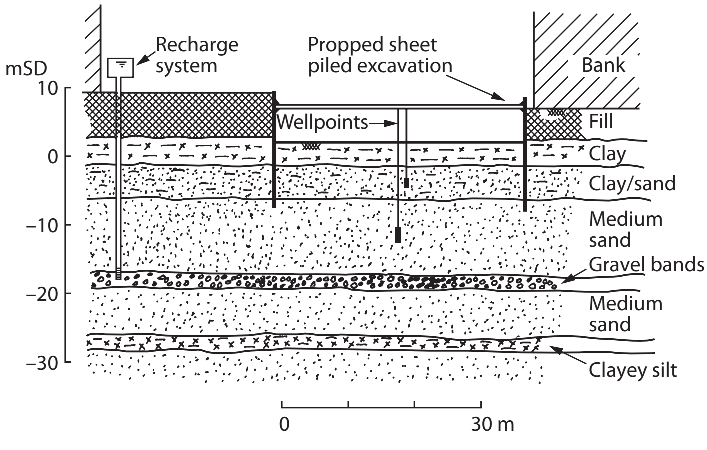
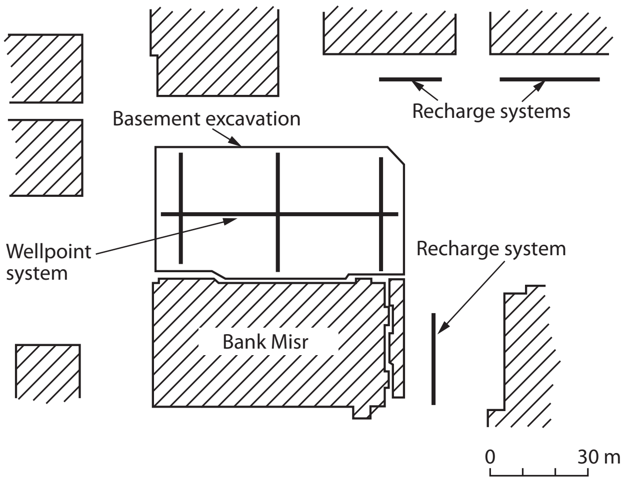


Figure 11.22 Bank Misr, Cairo—section through dewatering and recharge system. (Based on Troughton, V.M., *Groundwater Effects in Geotechnical Engineering*, Hanrahan, E.T., Orr, T.L.L., and Widdis, T.F., eds., Balkema, Rotterdam, 1987, pp. 259–264.)



Notes:
Recharge system consisted of recharge wells at 3 m centers
Pumping system consisted of wellpoints at 3 m centers

Figure 11.23 Bank Misr, Cairo: plan layout of dewatering and recharge system. (Based on Troughton, V.M., *Groundwater Effects in Geotechnical Engineering*, Hanrahan, E.T., Orr, T.L.L., and Widdis, T.F., eds., Balkema, Rotterdam, 1987, pp. 259–264.)

three lines, with wells at 3-m spacing. The recharge wells (which consisted of 125-mm-diameter screen and casing installed in a 250-mm-diameter borehole) were designed to make use of the soil stratification at the site by injecting water into the most permeable horizon present, a layer of gravel at 28 m below ground level. Water could be relatively easily injected into this permeable layer, from where it would feed to the other soil layers, analogous to the underdrainage dewatering effect (see Section 7.6) in reverse. The discharge water was fed to the recharge wells from a header tank 2 m above ground level; total flow rate was in the range of 25–40 L/s. An array of pneumatic piezometers (a type of rapid-response piezometer) was installed to monitor pore water pressure reductions in the compressible alluvial clays. Conventional standpipe piezometers were used in the sands.

In operation, the maximum drawdowns observed below structures were limited to 0.7 m, which resulted in settlements of 3 mm, around half the value predicted by calculation. As is often the case, the efficiency of the recharge wells reduced with time, and they had to be redeveloped by airlifting at monthly intervals to maintain performance. Two of the lines of recharge wells were located in a car park used during daytime business hours; these had to be redeveloped at night.

As mentioned earlier, one of the aims for the recharge system was to dispose of the discharge water without overloading the antiquated sewer system. However, as reported by Cashman (1987), the dewatering field supervisor did not have a lot of faith in recharge and tapped into the sewer system with a hidden discharge pipe. Thus, in fact, much of the discharge water was actually going down the sewer. Unfortunately, over the Christmas period, one of the main Cairo pumping stations broke down, and everything flooded back. The chairman of the main contractor received a telephone call from the mayor of Cairo Municipality, demanding his personal presence on-site immediately. He was told that, if this ever happened again, he, the chairman, would be immediately put in jail. This was a pretty convincing argument to ensure that things were put right and the recharge system stayed in operation.

11.9.8 Case history: Artificial recharge to protect drinking water source

Some of the challenges of artificial recharge systems can be illustrated by a case history from the United Kingdom in the 1990s. A large excavation was to be constructed to form the portal of a proposed road tunnel beneath a river. This involved dewatering of, and excavation into, an aquifer that was extensively exploited for public drinking water supply via high-capacity wells. In particular, one major public water supply well was located within a few kilometers distance of the site.

During planning of the dewatering, it was identified that the project posed two potentially significant risks to the aquifer and the public water supply well. First, in the location of the tunnel, the aquifer was confined by a thick sequence of Quaternary deposits of very low vertical permeability. This layer would be punctured by the excavation, meaning that a site pollution incident (such as a fuel spill) could result in pollution of the public supply aquifer (this type of problem is discussed in Section 15.7). Second, the pumping of groundwater during dewatering would lower groundwater levels in the aquifer over a wide area and could potentially affect the public supply well, reducing its yield. As a result, the public water supply in the region could have been detrimentally affected (this type of problem is discussed in Section 15.4.12).

The first risk was dealt with by good site management practices to minimize the likelihood of a site pollution incident in the areas where the aquifer was temporarily exposed. In addition, the permanent works were designed to ensure that they formed a seal through the Quaternary deposits to replicate the natural very low vertical permeability.

The second risk was mitigated in consultation with the environmental regulator, resulting in the use of artificial recharge. Unlike the case history in Section 11.9.7, where the objective was to minimize settlement around defined structures, in this case, the objective was to reduce net abstraction of groundwater from the aquifer. Therefore, there was no requirement to cluster the recharge wells around specific buildings or areas; instead, the aim was to return the water to the aquifer over a relatively wide area. By agreement with the environmental regulator, a defined flow rate (significantly less than the maximum dewatering pumping rate) could be discharged to the river, with the remainder returned to the aquifer via recharge wells.

This type of application of artificial recharge system allows some flexibility in the location of recharge wells. However, consideration must be given to the interaction between the dewatering and recharge system. The dewatering flow rate was substantial (in excess of 100 L/s). Numerical groundwater modeling showed that, if recharge wells were located within approximately 750 m of the dewatering wells, then a significant proportion of the recharged water would be drawn to the dewatering system, decreasing drawdown at the excavation and increasing pumped flow rates in a feedback loop.

Based on the numerical modeling, it was determined that the recharge wells should be located more than 750 m from the dewatering system around the tunnel portal. This raised the problem that this would require recharge wells in land not under the control of the construction project. This was addressed by making the best use of the land available to the project to develop four arrays of recharge wells (Figure 11.24a). The route of the proposed road was used as much as possible as a pipeline route to transmit the water from the dewatering system to the recharge wellfields. Some recharge wells were located in the verge of the partially constructed road (Figure 11.24b). Other recharge wellfields were located in land leased from local landowners.

The distance between the dewatering system and the recharge wellfields meant that it was not practicable or cost-effective to use the borehole submersible pumps to drive the water to the recharge wells. An array of booster pumps (Figure 11.25) was used to boost the water along pipelines to the recharge wells. In an attempt to reduce clogging by iron-related compounds (see Section 11.9.5), the system was operated as a "sealed system" to reduce contact between the pumped water and air. The use of a sealed system also had the advantage that it reduced the risk of external contamination affecting the recharge water. To reduce the risk of contamination of the aquifer and the public water supply wells, it was essential that the water recharged into the aquifer was uncontaminated, and this was checked by a program of regular chemical testing.

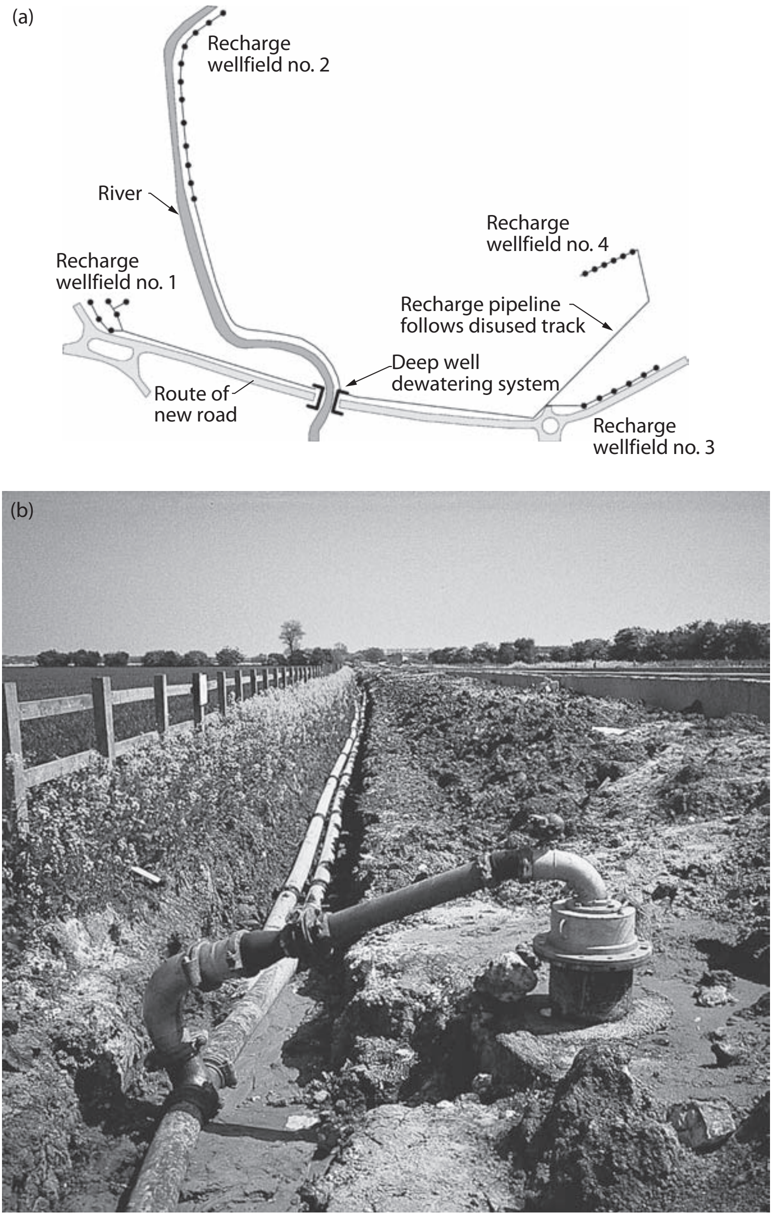


Figure 11.24 Location of recharge wells on road construction project. (a) Location of recharge wells. (b) Recharge wells located in verge of partially constructed road. (Courtesy of WJ Groundwater Limited, Bushey, U.K.)

Figure 11.25 Booster pumps use to drive water from dewatering system along transfer pipeline to recharge wellfield. (Courtesy of WJ Groundwater Limited, Bushey, U.K.)

The recharge system in operation exhibited typical performance characteristics of such systems. Over a period of weeks, the capacity of the recharge wells gradually reduced as a result of the clogging associated with the deposition of iron-related compounds. The type of clogging was clearly evidenced on-site by the red-brown effluent produced when the wells were cleaned by airlifting and backflushing. A regular program of rehabilitating wells by these methods was adopted, which typically restored each recharge well back to close to its original capacity for a few weeks. However, after a further few weeks of operation, the reduction in recharge capacity again became significant and the well was rehabilitated again. In each recharge wellfield, the process of well rehabilitation was almost continuous, with a crew scheduled to move sequentially from well to well in a regular cycle.

11.10 DEWATERING AND GROUNDWATER CONTROL TECHNOLOGIES USED FOR THE CONTROL OR REMEDIATION OF CONTAMINATED GROUNDWATER

Many construction projects are carried out on or near sites where there is a legacy of soil and groundwater contamination. Such contamination may result from current or former industrial uses, preexisting pollution incidents, or landfills and waste management sites. Groundwater pumping on or in the vicinity of such sites is likely to have an impact on the preexisting contamination, for example, by causing plumes of contaminated groundwater to move in different directions than otherwise. Such impacts must be assessed during the design process; this is discussed in Section 15.4.10.

Where the existence of groundwater contamination is known or suspected, the technology commonly used for groundwater control can be applied as part of a strategy to control or remediate contaminated groundwater. Methods based on groundwater pumping (i.e., sumps, wells, and artificial recharge) and groundwater exclusion (i.e., cutoff walls) can be used. Potential applications of these techniques on contaminated sites are described in the following sections.

Groundwater control technologies may be used on sites where contaminated groundwater is an issue in order to meet one or more of the following objectives:

1. Use of pumping systems to control or manipulate the movement of contaminated groundwater in preexisting plumes. In some circumstances, the pumping is intended to lower the groundwater levels around the contaminated area so that groundwater flow is toward the source, preventing outward migration of contaminants; this approach is sometimes called hydraulic containment (Figure 11.26a).
2. Use of pumping systems to extract and treat mobile contaminants, either from the source zone or from the surrounding contaminant plume; this approach is sometimes termed “pump and treat” (Figure 11.26b).
3. Use of pumping systems located close to a contaminant source (e.g., a leaking tank or silo) to attempt to intercept leakage before it has the opportunity to migrate away from the source (Figure 11.26c).
4. Use of pumping systems to lower groundwater levels in order to achieve secondary objectives. These objectives may include holding groundwater levels below a known contamination source zone, dewatering of excavations to allow removal of contaminated soils or structures, or lowering of groundwater levels to promote volatilization of contaminants for soil vapor extraction or increase bacterial access to oxygen as part of wider remediation strategies (Figure 11.26d).
5. Installation of low-permeability cutoff barriers to exclude or contain zones of contaminated soil or groundwater (Figure 11.26e).
6. Installation of buried passive treatment systems (often associated with sections of low-permeability cutoff barriers). These systems are known as permeable reactive barriers (PRBs; Figure 11.26f).

The design and execution of groundwater control on contaminated sites or the use of groundwater control methods as part of remediation schemes is complex. It is essential that the team developing any such scheme include designers and professionals experienced in contamination remediation.

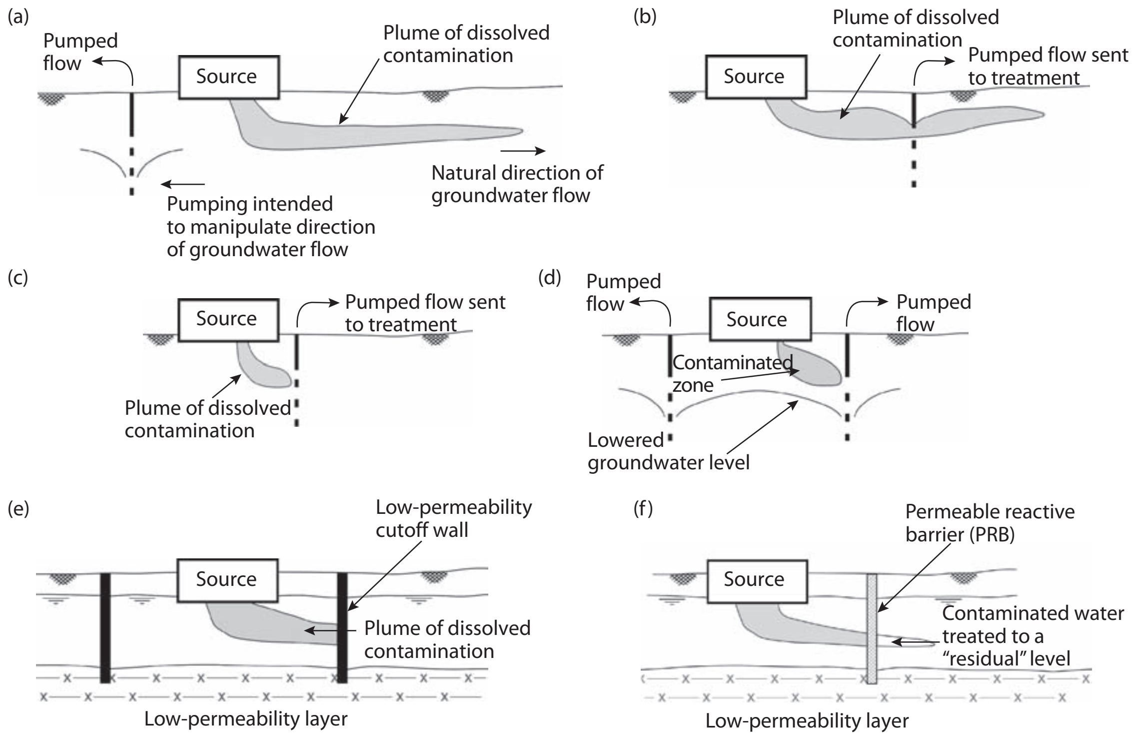


Figure 11.26 Groundwater control technologies used on contaminated sites. (a) Pumping systems to control the movement of contaminated groundwater. (b) Pumping system to extract and treat mobile contaminants (pump and treat). (c) Pumping systems to intercept leakage close to the source. (d) Pumping systems to lower groundwater levels to facilitate other processes. (e) Cutoff barriers to exclude or contain contaminated groundwater. (f) Permeable reactive barrier (PRB).

11.10.1 Applications of pumping systems on contaminated sites

In most cases, there are no fundamental differences in the groundwater pumping systems applied on contaminated sites compared to sites where contamination is not an issue: the physics of groundwater flow and the mechanics of pumping are the same in both cases. The key difference is that the water pumped is likely to contain elevated levels of contaminants and, therefore, is likely to require some form of treatment or specialized disposal route.

One case where groundwater pumping systems on contaminated sites may need to diverge slightly from the norm is where significant quantities of light nonaqueous phase liquids (LNAPL) exist as a layer of “free product,” floating on top of the water table. This situation can arise on sites contaminated by spills of petroleum and other light hydrocarbon compounds. If conventional well designs and pumping systems are used, there is a risk that the free product will become mixed with the pumped water, producing a hard to treat emulsion. A better approach is to provide larger diameter wells with screens that extend above the water table and provide two separate pumping systems in each well. The water pump would be set deep, below the free product level, and shallow “skimmer” or “scavenger” pumps would float on the water level in the well to directly draw off the free product. Methods suitable for use on sites where free product is present are given in the work of Holden et al. (1998).

In general, to produce an effective, efficient, and environmentally acceptable groundwater abstraction system, the design and construction of groundwater pumping systems on contaminated sites should take the following considerations into account, in addition to the general principles outlined elsewhere in this book:

1. The presence of suspended solids (fine particles of silt/sand/clay) in pumped water significantly increases the difficulties of treatment. It is almost unavoidable that sump pumping operations will generate water with significant suspended solids content. Therefore, to simplify treatment requirements, sump pumping should be avoided on contaminated sites whenever possible. Systems based on wells (such as wellpoints or deep wells) should be used instead, because these systems, when designed, installed, and developed appropriately, tend to produce water with low suspended solids content.
2. Peak water flow rate requiring treatment should be minimized. In many cases, the capital cost of a groundwater treatment system is more or less proportional to the peak flow rate. Reducing the flow rate of contaminated water will significantly reduce costs. Flow rates may be minimized by the use of groundwater cutoff barriers, by ensuring

that only the minimum necessary drawdowns are generated, and by segregating contaminated and "clean" water (which can be allowed to bypass the treatment process) so far as practicable.

3. Assuming that the site investigation has identified the extent and location of the contaminants, it may be possible to target dewatering wells (or screened sections within wells) directly on the most contaminated zones or strata. Ultimately, a system could be designed to have "clean" wells pumping uncontaminated water (which need not be treated) and a separately connected system of "contaminated" wells, perhaps pumping from a different depth or stratum, producing water that is then fed to a treatment plant.
4. Any wells should be designed appropriately so that they do not act as potential vertical migration pathways for pollution. It may be appropriate for wells to have grout seals above and below the well screens to reduce the risk of creation of artificial vertical flow paths (see Section 15.5).
5. Surface water should be managed efficiently. On most contaminated sites, there is a risk that any water allowed to run or pond on top of exposed soils will itself become contaminated and, therefore, potentially require treatment. Good surface water management practices (see Section 5.2) should be applied to keep surface water away from contaminated areas so far as possible. Any contaminated surface water should be segregated from "clean" surface water.

The most common way that pumped water containing elevated levels of contamination is managed is by being passed through an on-site treatment system (see Section 11.10.3) to reduce contamination levels before being discharged to a conventional water disposal route such as sewer or surface watercourse. In some cases, the treated water is recharged back into the ground via recharge wells or trenches.

On rare occasions when modest volumes of very highly contaminated or difficult to treat water are involved, the water may be transported off-site by road tanker to a specialized licensed disposal facility; this approach quickly becomes impractical if the volumes are more than a few hundred cubic meters. As an illustration, a large articulated road tanker will hold around 20 m^3 of water. Therefore, a minimum of five road tankers would be needed to transport every 100 m^3 of water for disposal. The cost and disruption of large numbers of road tanker movements militates against this approach on most projects.

Other chapters in this book give specific details on pumping systems such as sump pumping (Chapter 8), wellpoints (Chapter 9), deep wells (Chapter 10), and ejector wells (Section 11.2). Applications of groundwater pumping systems on contaminated sites are described in Holden et al. (1998).

11.10.2 Applications of barrier systems on contaminated sites

Barrier systems based on low-permeability cutoff walls are widely applied on sites where contaminated groundwater is a concern. If used to enclose a contaminated site, they have the obvious advantage of potentially isolating the contamination from the wider groundwater regime, either as a long-term strategy or to facilitate other remediation or construction activities. The most commonly used barrier methods on contaminated sites are slurry trench walls and sheet-pile walls, although other methods are used occasionally. An example of a slurry trench application on a contaminated site is given in Section 11.10.4.

It is important to recognize that no groundwater cutoff barrier is truly impermeable. Leakage should be expected through joints in the wall (see Section 12.3). Indeed, even a very low-permeability material has a finite permeability and will permit some seepage in the very long term directly through the mass of the barrier. In any event, the soil or rock within the area enclosed by the barrier will contain water that will need to be pumped out in advance of any excavation. Therefore, even extensive and high-specification barrier systems will not completely avoid the need to deal with contaminated water to some degree.

A further complication occurs when groundwater cutoff walls are used to enclose large areas. If the barrier formed by the cutoff wall is keyed into an underlying low-permeability stratum, the groundwater within the area enclosed by the walls will be trapped there; in fact, that is probably the objective of the barrier. The complication arises, because precipitation falling on the ground surface within the enclosed area is likely to infiltrate into the ground. If the rate of infiltration is significant, then it is possible that, over time (perhaps several years), groundwater levels will rise within the area enclosed by the wall. This may lead to problems with flooding of buried structures and services or waterlogging of the ground. If water levels rise, an outward hydraulic gradient will be created across the wall. Leaks or imperfections existing in the wall may promote leakage of contaminated water out of the area enclosed by the wall. One solution is to use long-term pumping to maintain the groundwater levels in the enclosed area slightly lower than groundwater levels outside the wall. This maintains an inward hydraulic gradient and prevents outward migration of contamination via groundwater flow. This approach was adopted in the case history described in Section 11.10.4.

A technique based on groundwater barrier technologies that is occasionally used to remediate groundwater migrating across or out of a site is the PRB. This approach accepts that the existence of an artificial barrier in the ground will result in hydraulic gradients across the barrier, even in the absence of pumping. In PRBs, all or part of the barrier is formed from a permeable material that can react with groundwater, removing contamination as groundwater flows through it as a result of natural groundwater flow (Figure 11.26f). Since the mid-1990s, PRB systems have been applied on sites where there is a desire to reduce levels of contamination in groundwater migrating from a site while avoiding the cost and maintenance issues associated with long-term pumping.

Specific details of the techniques used to form low-permeability barriers are given in Chapter 12. Applications of low-permeability barrier systems on contaminated sites are described in the work of Privett et al. (1996). PRBs are described in the work of Carey et al. (2002).

11.10.3 Groundwater treatment technologies

Where groundwater is pumped on contaminated sites, there is a potential requirement to treat the water prior to disposal. The requirement for treatment may be to reduce levels of contamination to levels acceptable for disposal to sewer (see Section 17.4), to meet regulatory requirements for disposal to surface waters or artificial recharge to groundwater (see Section 11.9), or to reduce environmental impacts resulting from discharge of groundwater (see Section 15.8).

A wide range of treatment methods is available. Oftentimes, a given contaminant could be treated by several quite different methods, applied in series; the choice of method will depend on the concentration of contaminants, the discharge flow rate, the duration of pumping, and the availability of treatment equipment and technologies. Some of the available technologies are described by Nyer (1992) and Holden et al. (1998).

The most commonly used groundwater treatment technologies are described as follows.

Buffer storage. Holding tanks or lagoons are normally provided upstream of the main treatment system to help smooth out any peaks or lows in flow rate or contaminant concentration. The buffer storage is normally sized to provide sufficient storage to allow the dewatering system to continue to operate while the treatment plant is offline (due to maintenance or breakdowns) for specified periods. The tanks or lagoons will also allow for some settlement of suspended solids in the pumped water.

Oil–water separation. The presence of even limited amounts of "free product" (nonaqueous phase liquids, immiscible with water; see Section 15.4.10) can cause problems with later treatment stages. An oil–water separator is often provided as one of the first stages of treatment to remove any free product from the treatment stream (the oil will require separate disposal).

Metals removal. The most common approach to metals removal is to promote the conversion of metals from soluble to insoluble form and then to settle out and remove the resulting precipitates. Typically, the pH of the water is raised (made more alkaline) by the addition of bases to reduce the solubility of metals. Coagulation and flocculation follow through the addition of specialist chemicals to promote precipitation. Finally, the precipitates are removed by a process of settling/clarification, where the settled solids are collected and removed as sludge, for further processing and disposal.

Filtration. Filtration by sand filters or bag or cartridge filters is sometimes carried out to reduce suspended solids in the treatment stream. Filters will require periodic cleaning, backwashing, or replacement as solids collect in them.

Granulated activated carbon (GAC) filtration. GAC has the ability to adsorb hydrocarbons and other compounds from liquid and vapor streams and is widely used to treat contaminated groundwater. The water is passed through tanks or vessels containing the GAC, and the contaminants sorb onto the surface of the GAC granules. The capacity of a given quantity of GAC to absorb contamination is large but finite and will eventually be exhausted. When that happens, the spent GAC is normally removed from site (for disposal or regeneration and later reuse) and replaced with fresh material. Typically, a treatment system would comprise two GAC vessels (primary and secondary) connected in series. Water quality is tested between the primary and secondary units. When the sorption capacity of the primary unit is exceeded, excessive levels of contaminants will be detected at the sampling point between the two units. When this occurs, the flow is redirected to the original secondary unit, while the GAC is replaced in the primary unit.

Air stripping. This technique is used primarily to remove volatile and semivolatile hydrocarbons from water. Such compounds are relatively easy to transfer from the liquid phase (water) to the vapor phase (as an off-gas). This is achieved by pumping the contaminated water to the top of an air stripping tower and allowing the water to cascade downward over a multitude of plastic packing elements within the tower. At the same time, a fan blows air upward through the tower. The air–water contact, distributed over the very large surface area of the packing elements promotes the stripping of the volatile compounds from the water, which leave the tower in the off-gas airflow. The off-gas is often treated by passing through vapor-phase GAC to reduce air emissions. Air stripping is often used in series with GAC filtration. If the air stripper is located upstream of the GAC filter, the volatile hydrocarbons will be removed before the GAC filter. This reduces the contaminant loading on the GAC filter and will probably increase the life of the GAC media.

Figure 11.27 Modular groundwater treatment plant used to treat discharge from dewatering system. (Courtesy of Hölscher Wasserbau GmbH, Haren, Germany.)

For most construction-related groundwater control or remediation projects, the groundwater treatment system will be required for a period of several months, perhaps up to a maximum period of 1–2 years. In these applications, groundwater treatment plants commonly comprise temporary systems of modular steel tanks and vessels (Figure 11.27) delivered to site and then commissioned in suitable configurations to meet the project requirements. For very long-term remediation projects, it may be cost-effective to construct groundwater treatment plants to similar standards to permanent wastewater treatment systems (this approach was taken for the case history in Section 11.10.4).

11.10.4 Case history: Cutoff walls and groundwater abstraction at Derby Pride Park

During the 1990s, extensive contamination remediation works were carried out on a 96-ha site, close to the center of Derby, U.K. (Braithwaite et al. 1996; Barker et al. 1998). The largely derelict site had formerly been used for domestic/industrial landfill, coke, gas, and heavy engineering works and gravel extraction. It was heavily contaminated with pollutants, including oils, tars, heavy metals, phenols, and ammonium, and boron. The site was remediated on behalf of Derby City Council, with Arup acting as designers. The remediated site was to be known as Derby Pride Park.

It was intended to reclaim the site for mixed commercial, leisure, and residential use. The remediation strategy included an extensive groundwater cutoff wall and associated groundwater abstraction system.

Extensive ground investigations indicated that ground conditions comprised fill and alluvium over highly permeable terrace gravels, which were in turn underlain by lower-permeability Mercia Mudstone. Extensive areas of soil contamination existed on the site, and groundwater contamination was detected in the terrace gravels. There was concern that existing contamination could migrate from the site and enter an adjacent river.

As part of the wider remediation strategy, a groundwater cutoff wall was constructed around the most contaminated area of the site. The solution finally adopted was a slurry trench wall (see Section 12.6) approximately 3 km long and a maximum of 10 m deep, around the perimeter of the identified area, to create an enclosing low-permeability barrier, cutting off the fill, alluvium, and terrace gravels and keying into the Mercia Mudstone by 1 m (Figure 11.28).

The wall was constructed as a 600-mm-thick cement–bentonite slurry trench wall, dug by backactor. The slurry mix design comprised cement, bentonite, and ground-granulated blast-furnace slag (GGBS) with a target permeability of 1×10^{-8} m/s at 28 days. In addition, an HDPE membrane with a design permeability of 1×10^{-9} m/s was placed within the wall. The membrane was installed in panels using a special handling frame, each panel being connected to its neighbors using prefitted interlock joints. The wall alignment crossed 36 preexisting underground services (pipes, sewers, and ducts) that had to be incorporated through the wall (via welded seals in the HDPE membrane) without compromising design permeability. Once the wall was at least 7 days old, the top 0.5 m was dug out and replaced with a clay cap.

Numerical groundwater modeling at the design stage predicted that the presence of the cutoff walls would have a modest impact on external

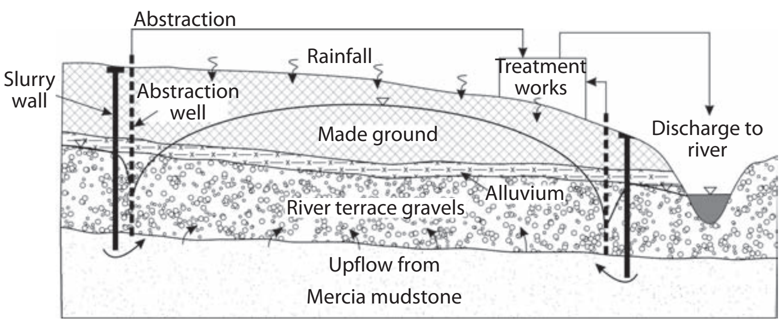


Figure 11.28 Schematic section through the Derby Pride Park Remediation Project. (Based on Braithwaite, P. et al., *Arup Journal*, 31(1), 13–15, 1996. With permission.)

groundwater levels. The barrier effect of the wall was predicted to increase external groundwater levels by approximately 0.7 m on the upgradient side of the site.

Groundwater modeling also predicted that, without control measures, groundwater levels within the area enclosed by the wall would rise, partly due to infiltration from surface precipitation and partly due to upward seepage of groundwater from the underlying Mercia Mudstone. The solution adopted was to install an array of groundwater abstraction wells to maintain groundwater levels inside the wall (approximately 0.25 m) slightly lower than external groundwater levels. In addition to preventing the problems associated with groundwater level rises within the site, this solution had two other advantages. First, it provided an element of "hydraulic containment" so that, if there was a leak or breach in the wall, water would flow into the site, rather than contaminated water flowing out. Second, pumping of the contaminated water from within the site would, over time, remove much of the dissolved contamination from the site, resulting in a gradual cleanup of contamination levels.

The groundwater abstraction wells were connected via a ring main to a water treatment plant located in one corner of the site. The water was treated to a standard determined by the regulatory permissions for water discharge at the site, and the treated water was then disposed of to the river. The groundwater abstraction and treatment system was intended to operate for 15 years. Due to the relatively long design life, the treatment plant was not of the temporary modular tank type commonly used on groundwater control projects, but was, instead, constructed to similar standards to permanent wastewater treatment plants.

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Chapter 12

Methods for the exclusion of groundwater

12.1 INTRODUCTION

On many belowground construction projects, it may be necessary to deploy ground engineering techniques to exclude groundwater from an excavation. As discussed in Chapter 5, these exclusion methods may be deployed on their own or in combination with dewatering pumping techniques.

Although the principal focus of this book is on groundwater control techniques based on pumping technologies, it is appropriate for dewatering practitioners to have at least a basic understanding of exclusion methods used to form cutoff barriers or walls. This knowledge will be useful when comparing pumping and exclusion options or when developing methods to mitigate environmental impacts (see Chapter 15). This chapter provides a brief overview of the more commonly used groundwater exclusion techniques, including the factors affecting the application of each technique. The systems described in this chapter are

1. Steel sheet piling
2. Vibrated beam walls
3. Slurry trench walls
4. Concrete diaphragm walls
5. Bored pile walls
6. Grout barriers
7. Mix-in-place barriers
8. Artificial ground freezing

12.2 PRINCIPAL METHODS FOR GROUNDWATER EXCLUSION

A wide range of exclusion techniques are available to form cutoff walls or barriers around civil engineering excavations. Those commonly used for groundwater control purposes are summarized in Table 5.1. They have been categorized by the way each method interacts with the ground. Displacement barriers (such as steel sheet piling or vibrated beam walls) are inserted into the ground, displacing soil and, consequently, generating little or no spoil. In contrast, excavated barriers (such as slurry trenching, concrete diaphragm walls, and bored pile walls) involve excavating material from the ground (thereby generating spoil) and replacing it with barrier material. Injection methods (the various forms of grouting) involve injecting and/or mixing special fluids into the ground, where they solidify to block groundwater flow in pores and fissures. Another category of exclusion methods is the system of artificial ground freezing where the ground is modified by thermal means. The ground is frozen to form a barrier of very low-permeability frozen ground and groundwater.

The selection of a given exclusion method used to form a cutoff barrier will depend on the conditions and constraints on a given project. Primary constraints are the desired depth of wall, ground conditions, geometry of wall (some methods can be used horizontally or inclined to the vertical, whereas other methods are limited to vertical applications), and whether the barrier is intended to be permanent or temporary (temporary barriers can be useful in reducing long-term groundwater impacts, as discussed in Section 15.6). The various methods are described in the remainder of this chapter. Applications of exclusion methods specific to tunneling are described in Section 5.6.2.

There may be occasions when different exclusion methods are used in combination. In the same way that more than one pumping technique may be used if multiple aquifer systems are present, exclusion methods may also be combined to deal with the particular stratification on a site. Soudain (2002) describes a project where a 2.23-km-long groundwater cutoff wall was retrofitted around a large waste disposal pit used to dispose of half a million animal carcasses following a foot-and-mouth disease outbreak in the United Kingdom. In that case, a jet-grouted barrier was first formed in the fractured bedrock at the site. Then, a slurry trench wall was excavated through the overlying drift, into the weathered bedrock, and was keyed into the upper 1.5 m of the jet-grouted zone to provide a continuous low-permeability barrier to a greater depth than could have been achieved by slurry trenching alone.

12.3 GEOMETRIES OF EXCLUSION APPLICATIONS

The application of exclusion methods to provide a physical cutoff wall around an excavation is described in Section 5.4 and Figure 5.2. In some applications where there is a requirement to provide a temporary or permanent retaining structure (e.g., to support the sides of an excavation or to form the walls of a permanent basement), the groundwater cutoff barrier may also act as a structural element of the permanent works. This will affect the choice of technique used to form the cutoff wall, because not all methods are suitable for structural applications.

The most common application is to install vertical cutoff walls around an excavation. However, some exclusion methods, including grouting and artificial ground freezing, can be installed in nonvertical geometries to produce various cutoff geometries, and some methods can be used to form low-permeability plugs to seal the base of excavations (Figure 12.1).

One of the features of most methods used to form cutoff walls is that they are typically not installed continuously but, for practical reasons, are installed in discrete elements. These elements (typically panels or columns) are intended to overlap or intersect to form a continuous wall of very low-permeability material (Figure 12.2). Accordingly, a key element of

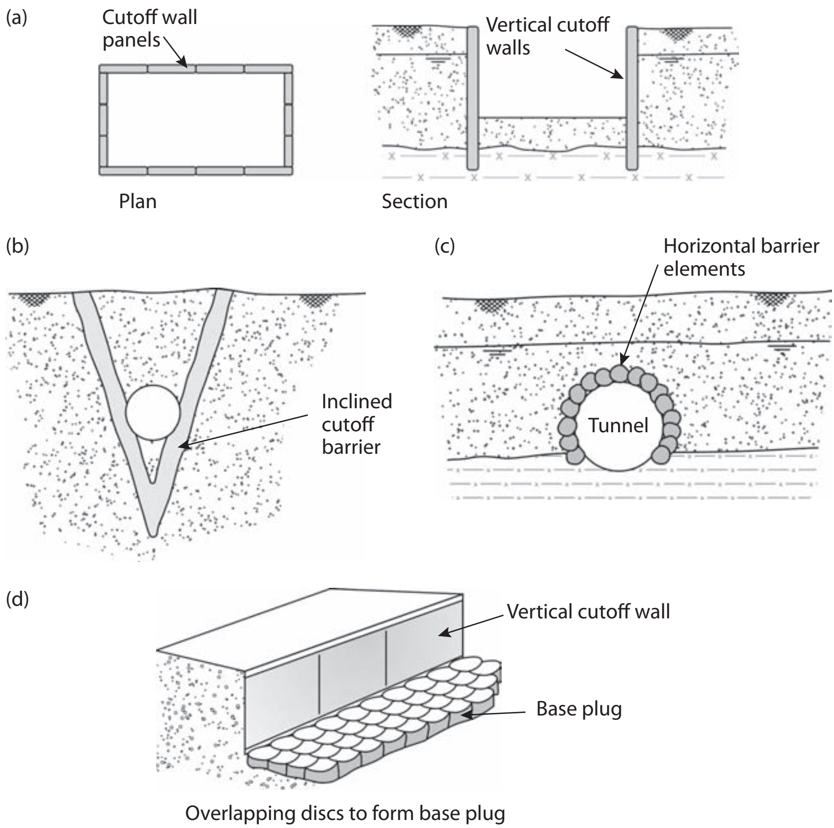


Figure 12.1 Typical geometries for cutoff barriers. (a) Vertical barriers. (b) Inclined barriers. (c) Horizontal barriers. A horizontal barrier, arranged to exclude groundwater from a tunnel, is located at the interface between higher and lower-permeability strata. (d) Base plugs. A base plug consists of overlapping columns of ground treatment.

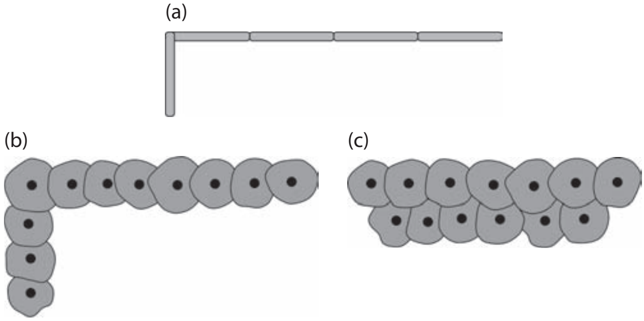


Figure 12.2 Elements used to form cutoff walls. (a) Intersecting panels or structural elements. (b) Overlapping columns or piles. (c) Multiple rows of overlapping columns.

the design and construction of a cutoff wall is ensuring the integrity of the joints between the elements in the wall. When, as occasionally happens, cutoff walls do leak significantly, the most likely cause is seepage through one or more joints. Leakage through joints is often caused by the deviation of alignment of adjacent panels or columns in the cutoff wall. As the panels deviate, they may cease to intersect, leaving a gap through which water can pass.

Howden and Crawley (1995) document the example of the extensive concrete diaphragm wall around Sizewell B Power Station in Suffolk, U.K., where monitoring of groundwater levels during dewatering indicated the presence of a leak, which required sealing by grouting (see Section 5.7.2).

12.4 STEEL SHEET PILING

Steel sheet pile walls consist of a series of interlocked steel sections (typically of a "Z" or "U" profile) that are driven or pushed into the ground to form a continuous barrier (Figure 12.3). Sheet pile walls can be used (subject to appropriate design and/or the provision of propping) in a structural role to support the sides of an excavation, as well as to act as a barrier to groundwater flow. An important characteristic of sheet pile walls is that they can potentially be used to provide temporary cutoffs. If the sheet pile wall does not form part of the permanent works, it may be possible to remove the sheet piles during the later stages of construction, thereby reducing the potential groundwater impacts if a permanent groundwater cutoff barrier is left in place. Extracted sheet piles will commonly be used on subsequent construction projects; thus, any given sheet pile may be used to provide cutoff walls on several projects during its life.

Traditionally, sheet pile walls are installed by driving of the piles, where a piling hammer or vibrator is suspended from a service crane and attached

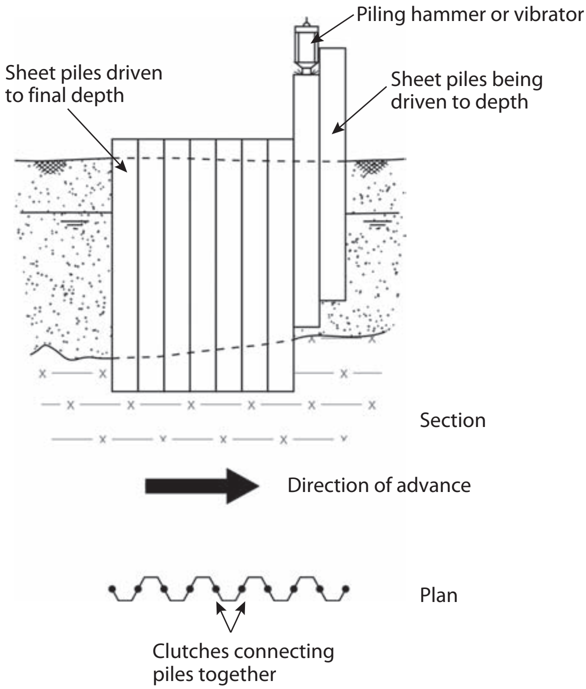


Figure 12.3 Construction sequence for installation of steel sheet piling.

to each pile in turn to drive it into the ground. These methods are still in widespread use, but it is recognized that they generate levels of noise and vibration that may not be acceptable, especially in urban areas or where sensitive structures are present. In recent years, there have been great advances in so-called "silent" methods of piling installation. These methods "press" the piles into the ground by the application (in stages limited by the throw of the jacks) of steady pressure from hydraulic jacks, rather than the dynamic forces from impact or vibration. The hydraulic pile presses used for this task use the reaction supplied either from previously installed piles (Figure 12.4) or from a large base machine. Such methods generate significantly lower levels of noise and vibration compared to traditional driving methods.

As previously discussed, the ability of a cutoff wall to exclude groundwater is highly dependent on the watertightness of its joints. A key issue with sheet piles is that, because of the narrow width of an individual sheet pile (typically 400–600 mm, measured in the direction of the wall), very many piles are needed to form a wall of any length and, consequently, there will be very many joints in the wall. On a typical sheet pile section, either end will be formed into a hook profile known as a "clutch." When used in a cutoff wall, the clutch of each pile slides into the clutch of its neighbor

Figure 12.4 Installation of steel sheet piling by the press-in method using hydraulic jacks. The hydraulic jacking unit sits on top of the previously driven piles. The jacking unit grips onto the piles and presses new piles into the ground using the resistance of the neighboring piles. (Courtesy of Giken Europe BV, London.)

on either side and, in theory, provides both a structural interlock and a relatively watertight seal. In reality, the clutches may leak significantly, and several proprietary sealants are available and can be installed in the clutches prior to driving, in an attempt to reduce leakage through clutches. It should be recognized that, if pile installation and alignment is not carefully controlled or if driving conditions are difficult, it is possible that piles may "declutch" during installation. If this occurs, part of the pile depth separates from its neighbors, which may leave a large permeable "window" in the cutoff wall through which water can flow.

A variant on sheet pile walls is the combined pile or "combi-pile" wall (Figure 12.5). Such walls comprise primary elements formed of large diameter heavy steel sections (often tubular steel piles) driven in a line, at a spacing such that gaps remain between them. These gaps are filled by secondary elements typically comprising steel sheet piles, which are linked to the primary elements by clutches. These walls are appropriate when the structure loading on a wall is high; the primary elements provide the principal

Figure 12.5 Combi-pile walls. The wall comprises primary elements (large-diameter tubular steel piles) and secondary elements (steel sheet piles) that are linked together by clutches.

structural strength and the secondary elements complete the wall, allowing it to act as a groundwater barrier.

Further information on steel sheet piling can be found in the work of Williams and Waite (1993) and BS EN 12063:1999.

12.5 VIBRATED BEAM WALLS

Vibrated beam walls involve successive overlapping insertions of a steel probe or mandrel into the ground and the injection of grout (typically a cement–bentonite slurry) into the slot left behind by the mandrel as it is withdrawn. The mandrel used to form the wall typically comprises a heavy-duty steel "H" beam, driven into the ground by a vibrator, hence the generic term vibrated beam walls. The method is known by a variety of other names, including thin grouted membrane, ETF wall, or vibwall, depending on the application and the company carrying out the work.

A typical installation sequence for a vibrated beam wall is shown in Figure 12.6. A vibrator is mounted atop the H beam, operated on a leader suspended from a service crane. A grout pipe is fixed to the web of the H beam to feed an injection nozzle located close to the base of the beam. The vibrator drives the beam to the target depth (grout may be injected at this stage to aid driving). In favorable conditions, depths of 20 m may be achieved, but penetration difficulties may be caused by the presence of cobbles or boulders. Preboring is sometimes used to loosen the ground prior to beam insertion. As the H beam is withdrawn, grout is injected down the pipe to fill the void left by the beam. Once withdrawn, the beam is inserted into the ground to form the next section of wall, ensuring that there is sufficient overlap to provide continuity of the wall. The whole process is repeated until the length of wall is completed. A section of vibrated beam wall exposed during excavation is shown in Figure 12.6c.

A vibrated beam wall has little structural value, and the principal groundwater control applications have been used to form deep temporary cutoff walls through easily penetrated ground conditions such as sands. A typical example was for the Conwy Crossing Project in the United Kingdom, where a vibrated beam wall was used to form a cutoff wall through a bund of hydraulically placed sand fill around a temporary casting basin used to construct elements for an immersed tube tunnel (Powrie and Roberts 1990). In that application, the bund was to be dredged out in the latter stages of the project to allow the tunnel units to be floated out. The modest structural strength of the wall was an advantage, because it did not impede dredging of the bund.

Because of the relatively narrow width of the majority of the wall (which is formed by the void left by the web of the H beam), there is a risk that deviation or deflection of the beam will result in the loss of overlap between adjacent beam insertions. If this occurs, water may be able to pass through

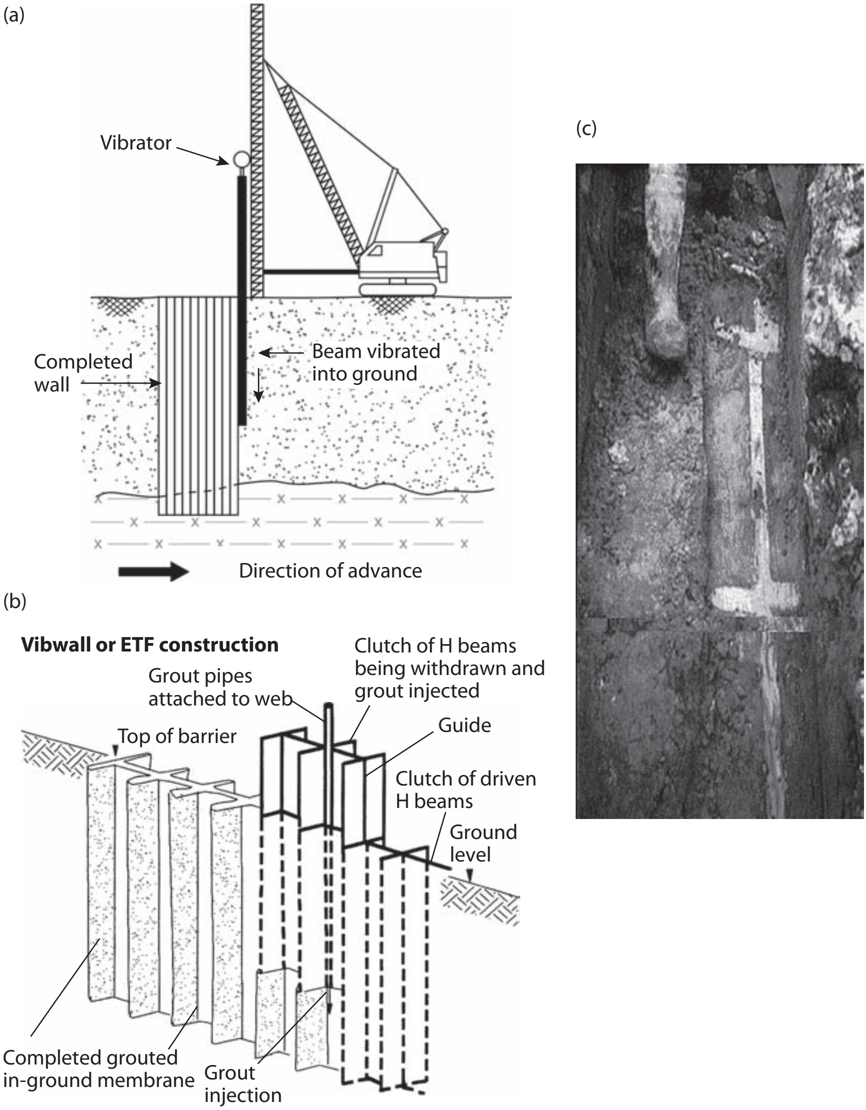


Figure 12.6 Construction sequence for vibrated beam cutoff walls. (a) Installation sequence for vibrated beam cutoff walls. (b) Schematic detail of vibrated beam cutoff walls. (From page 62 of Woodward, J., *An Introduction to Geotechnical Processes*, Spon Press, London, 2005. With permission.) (c) Exposure of vibrated beam wall during excavation. (Courtesy of Bachy Soletanche, Burscough, U.K.)

the wall. In some instances, clutches of two or three beams are driven together in an attempt to reduce the risk of deviation.

Further information on vibrated beam walls is given in the work of Privett et al. (1996) and McWhirter (2000).

12.6 SLURRY TRENCH WALLS

A slurry trench is formed by the excavation of a trench supported by a slurry (most commonly but not always bentonite) during excavation. Depending on the type of construction, a wall formed of soil–bentonite mixture or a self-hardening cement–bentonite mixture is produced. In its common forms, a slurry trench has limited lateral strength and is not intended to act as a structural wall for either temporary or permanent works, unless reinforced with support elements such as beams or steel cages. Its primary objective on civil engineering projects is to act as a barrier to groundwater flow; if the wall is to act as a retaining structure, a concrete diaphragm wall (see Section 12.7) may be more appropriate. On contaminated sites, slurry trench walls (often in combination with high-density polyethylene (HDPE) membranes set into the wall) are used to prevent subsurface migration of contaminants (Privett et al. 1996). A case history of a slurry trench wall used on a contaminated groundwater is described in Section 11.10.4.

The most common form of excavation is by continuous excavation by a backactor fitted with an extended boom and dipper arm (Figure 12.7a), which is effective for depths down to 15–20 m. Specialist long-reach backactors may work at depths of 25–30 m (Figure 12.7b). The excavator digs a trench that is kept topped up with bentonite or cement–bentonite slurry (Figure 12.7c). During excavation, the role of the slurry is to support the trench and prevent collapse. The level of slurry in the trench is kept topped up above groundwater level. This allows a thin "filter cake" of bentonite to form on each face of the trench as clay particles are filtered from the slurry as it seeps into the surrounding soil. The filter cake reduces the loss of slurry from the trench and allows a differential head to develop between the slurry and the groundwater outside the trench. This differential head of slurry plays a key role in maintaining trench stability during excavation, and typically, these walls can only be constructed if there is a minimum of 1–2 m head of slurry above natural groundwater level.

Soil–bentonite walls are most commonly used in countries influenced by American construction practices, whereas cement–bentonite walls are common in European-influenced areas.

Soil–bentonite walls are constructed by the excavation of a trench under bentonite slurry. A mixture of soil and bentonite is prepared adjacent to the trench and is then typically pushed into the trench from one end to form a steadily progressing shallow slope of fill material that tracks the

Figure 12.7 Construction slurry trench wall by backactor. (a) Excavation by conventional backactor fitted with extended boom and dipper arm. (Courtesy of Arup, London.) (b) Excavation by specialist long-reach backactor. (Courtesy of Inquip Associates, Inc., Santa Barbara, CA.)

(c)

Figure 12.7 (Continued) (c) Trench kept topped up with bentonite slurry during excavation. (Courtesy of Inquip Associates, Inc., Santa Barbara, CA.)

progress of the trench excavation (Figure 12.8). The material used for the fill is commonly spoil from the trench, although if the natural ground has limited fines content, imported fill may be used to give better wall properties. The mixing of the soil and bentonite at the surface is normally done by a bulldozer tracking and blading clean bentonite slurry into small stockpiles of soil. Where some limited structural strength is needed, cement may be added to the backfill mix to produce a soil–cement–bentonite wall.

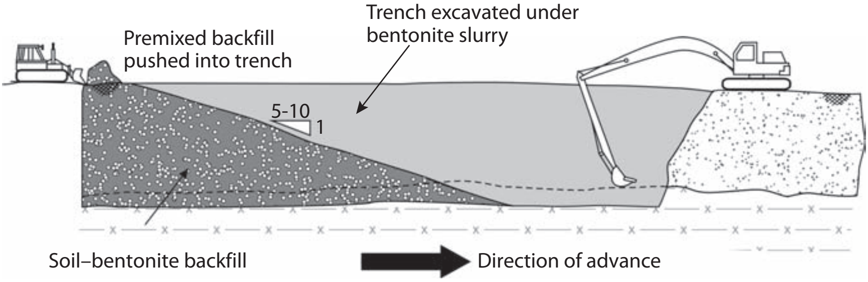


Figure 12.8 Construction sequence for soil–bentonite slurry trench wall.

Cement–bentonite walls can be constructed by either single- or two-stage construction:

1. In single-stage construction, the trench is excavated under the self-hardening cement–bentonite slurry, which later sets as the permanent backfill (Figure 12.9a). This approach may be satisfactory for shallow trenches where the excavation time is much less than the slurry setting time.
2. In two-stage construction, the trench is supported by a bentonite slurry during excavation, which is then replaced with the permanent cement–bentonite mix to form the final wall (Figure 12.9b). In this approach, temporary stop ends are used to separate the bentonite slurry used in excavation from the permanent cement–bentonite mix. This is beneficial to reduce the risk of cross-contamination affecting the properties of both materials.

The very top of the slurry trench wall may experience cracking due to drying out and is sometimes dug out and replaced with compacted clay

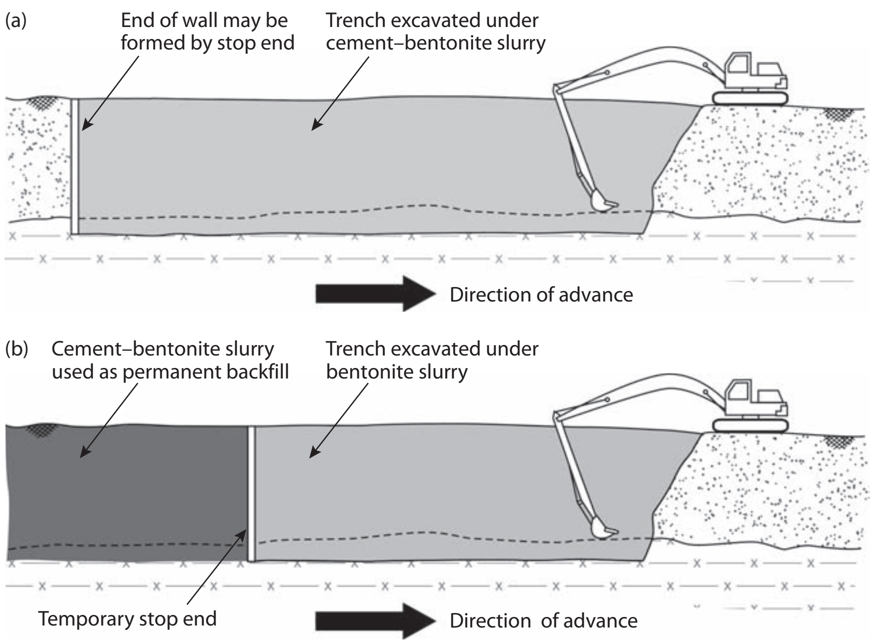


Figure 12.9 Construction sequence for cement–bentonite slurry trench wall. (a) One-stage construction for cement–bentonite wall. (b) Two-stage construction for cement–bentonite wall.

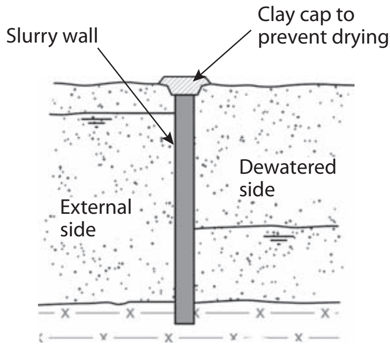


Figure 12.10 Capping of slurry trench wall with compacted clay.

(Figure 12.10) and may incorporate a shallow depth of HDPE membrane, particularly where the wall is to have a long life.

Although excavation by backactor is, by far, the most common method of construction for slurry trench walls and can achieve depths of 15–20 m using widely available equipment (and depths of 25 m or deeper using specialist backactors), other methods are also used. Draglines are occasionally used and are effective down to a 25-m depth of a wall. On rare occasions, specialist trenchers have been used (Brice and Woodward 1984; Schünmann 2004) and have been used down to 8-m depth. Deeper walls (down to 100-m wall depth) require construction by the noncontinuous panel method using diaphragm walling grabs or hydromills (see Section 12.7). Slurry trench walls to such depth are used only rarely in civil engineering applications and are more relevant to deep cutoffs for dam construction or to reduce seepage into open pit mines.

Typical wall widths are in the range of 300–3000 mm. Wall permeabilities of 10^{-7} to 10^{-9} m/s can be achieved. Typical target permeabilities in performance specifications are of the order of 10^{-8} to 10^{-9} m/s for cement–bentonite walls (without the use of polyethylene membranes); soil–bentonite walls typically produce permeabilities of the order of 10^{-9} m/s. The required thickness of the wall is determined to ensure that the hydraulic gradient across the wall is not excessive or to provide an adequate design life in aggressive ground conditions. In groundwater control applications, the hydraulic gradient is normally limited to 10–30.

Approximate mix design for the slurry is an important part of slurry trench wall work and should be carried out by specialists. Typically, a cement–bentonite slurry will comprise a mixture of various proportions of bentonite, ordinary Portland cement (OPC), ground-granulated blast-furnace slag to reduce permeability, and pulverized fuel ash (PFA) to reduce shrinkage. Typical slurry density will be approximately 1.1 tonne/m^3.

To ensure the stability of the trench during excavation, the head of the bentonite slurry must be maintained 1–2 m above groundwater level. Where groundwater level is close to ground level, it may be necessary to build up ground level by the placement of fill to ensure that the slurry head can be increased to an adequate level.

Because slurry trench walls are typically constructed by either continuous excavation or by the panel method, they are not particularly suitable when forming cutoff walls of complex plan layouts. They are most suitable for forming cutoff walls, where the plan layout is essentially rectangular or polygonal, with relatively long runs of wall between corners.

Further information on slurry trench walls is given in the work of Jefferis (1993) and the Institution of Civil Engineers (1999).

12.7 CONCRETE DIAPHRAGM WALLS

Diaphragm walls (commonly known as D-walls) are concrete walls, typically cast in situ, formed within a trench supported by a slurry (most commonly but not always bentonite) during excavation. The concrete is placed into the trench, displacing the slurry and forming a concrete wall in direct contact with the ground. In typical applications, the wall is constructed from reinforced concrete to allow it to act as a structural element of the permanent or temporary works. Less commonly, the wall may be intended purely as a groundwater cutoff barrier, with no structural role, and may be formed from unreinforced plastic concrete (plastic concrete is a relatively ductile concrete formed from aggregate, cement, and bentonite).

Typically, the construction sequence for a diaphragm wall involves the wall being excavated and concreted in discrete "panels" (Figure 12.11). Alternate primary panels are constructed, with the end of each panel formed by a temporary steel section known as a "stop end." Later, the stop end is removed by a crane, and the secondary panels are constructed between the primary panels. Typically, wall widths are 800–1500 mm (although thicknesses of up to 2500 mm have been installed), and panel lengths are 3–6 m.

Diaphragm walls are constructed by specialist excavating equipment, normally suspended from a heavy-duty crane or base machine, which form the trench within which the wall will be constructed. Excavation is carried out through surface "guide walls," which ensure positional accuracy and prevent the top of the wall from enlarging as a result of the insertion and removal of the digging tools. To prevent the trench from collapsing during excavation, it is kept flooded with bentonite slurry to a level above the external groundwater level. The hydrostatic pressure of the slurry on the trench faces improves the stability of the trench similar to slurry trench walls (see Section 12.6). The slurry will become contaminated with sediment during

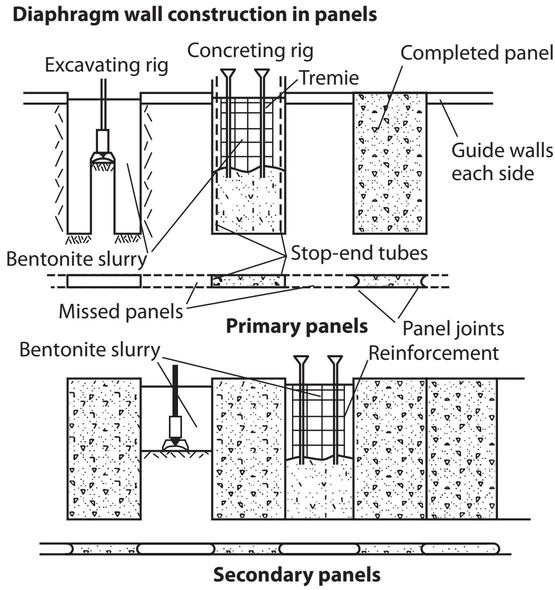


Figure 12.11 Construction sequence for diaphragm wall. (From page 53 of Woodward, J., *An Introduction to Geotechnical Processes*, Spon Press, London, 2005. With permission.)

the excavation process and is typically pumped from the trench, passed through a desanding plant, and recycled back into the trench.

When panel excavation is complete and residual sediment has been removed from the bentonite slurry in the trench, the concrete can be placed. For reinforced concrete walls, a prefabricated reinforcement cage is lowered into the slurry-filled trench. A specially designed concrete mix is then tremied into the trench. Concrete placement is carefully controlled to ensure continuous placement from the bottom up to displace the bentonite slurry from the trench.

Excavation for diaphragm walls is usually carried out by either grabs or hydromills. Because the trench is kept topped up with bentonite slurry, each time the digging tool is lowered into the trench, it disappears beneath the slurry; the plant operator is therefore effectively digging "blind." Great care must be taken to ensure the verticality and alignment of the panels to ensure that wall continuity is achieved over the full depth. Concrete guide walls at ground level play a key role as positional guides for the panel excavation.

The traditional excavation equipment for diaphragm walls is a heavy-duty grab (Figure 12.12a), suspended either on a cable or kelly bar. Grabs are suitable for excavation in soils and some soft rocks. Since the 1980s, there

Figure 12.12 Excavation equipment for diaphragm walls. (a) Diaphragm walling grab. (Courtesy of Bachy Soletanche, Burscough, U.K.) (b) Hydromill diaphragm walling rig. (Courtesy of Cementation Skanska Foundations, Maple Cross, U.K.) (c) Close-up of rotating cutters on hydromill. (Courtesy of Cementation Skanska Foundations, Maple Cross, U.K.)

has been considerable development in hydromills (Figure 12.12b). These units are sometimes referred to as rockmills or hydrofraises (after one of the first units developed by Soletanche in the 1980s). Rotating drum cutters (Figure 12.12c) are used to break up the soil or rock in the base of the excavation, which is then pumped with the slurry via a reverse circulation system to a desanding plant on the site. The clean slurry is then pumped back to the panel excavation. Hydromills typically allow faster excavation rates than grabs and have been effective in very stiff soils and in rocks, where excavation by grab would be slow and problematic. Diaphragm walls have been installed to depths of more than 120 m.

In order to ensure that the trench is adequately supported during excavation, the head of slurry must be maintained at least 1–2 m above groundwater level. Occasionally, dewatering systems such as wellpoints have been used to temporarily lower groundwater levels during wall excavation to ensure that sufficient slurry pressures are maintained in the trench.

Because diaphragm walls are typically constructed by the panel method, they are not particularly suitable for use in cutoff walls of complex plan layouts. They are most suitable for forming cutoff walls where the plan layout is essentially rectangular or polygonal, with relatively long runs of wall between corners. If complex plan geometries are required, bored pile walls may be more appropriate. Leakage occasionally occurs at panel joints (see the case history for the Sizewell Power Station in Section 5.7.2) and may require retrospective sealing.

Further information on diaphragm walling can be found in the work of Puller (2003) and BS EN 1538:2010.

12.8 BORED PILE WALLS

Bored pile walls are formed from circular concrete piles installed in close proximity (in some cases, intersecting with each other) to form a line to support the perimeter of the excavation and, potentially, to act as a barrier to groundwater flow. There are two principal configurations of bored pile walls.

1. *Secant pile walls*. In this application, alternate piles are drilled at spacings of less than one pile diameter so that they intersect and form a continuous wall (Figure 12.13a).
2. *Contiguous pile walls*. In this application, successive unconnected piles are bored in close proximity. Typically, a small gap remains between piles and may be sealed by grouting or other means (Figure 12.13b).

Bored pile walls have been used in groundwater control applications to form the support structures for conventional excavations and for circular and

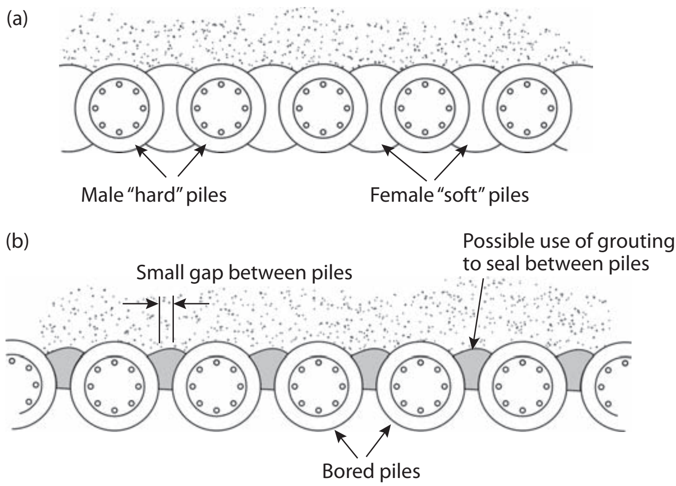


Figure 12.13 Bored pile walls. (a) Secant pile wall. Adjacent piles intersect to form a continuous wall. Female piles are installed first, and male piles are subsequently installed, cutting into the female piles. (b) Contiguous pile wall. Adjacent piles do not intersect, and a small gap remains between piles.

elliptical shafts. Because the wall is formed from individual piles whose relative position can be carefully controlled, bored pile walls are better able to provide walls of complex plan arrangements (curved walls, or complex intersections) than wall systems such as diaphragm walls that are installed in larger panels.

Additional information on bored pile walls is given in the work of Puller (2003) and BS EN 1536:2010.

12.8.1 Secant pile walls

In this application, bored cast in situ concrete piles (typically 600–1200 mm in diameter) are designed to intersect to form a continuous wall that can act as a groundwater barrier, as well as to form a structural wall, which can potentially form part of the permanent works.

Typically, the construction sequence involves the installation of alternate piles (known as primary or "female" piles) in each line and the subsequent installation of a secondary or "male" piles to fill in the gaps. The piles are spaced at less than one pile diameter so that the male piles cut into the female piles in a process known as "overcutting." The male piles cut a secant out of the female piles, thereby creating a continuous wall (Figures 12.13a and 12.14). The male piles are typically reinforced to provide the structural capacity of the wall, the intervening female piles being unreinforced.

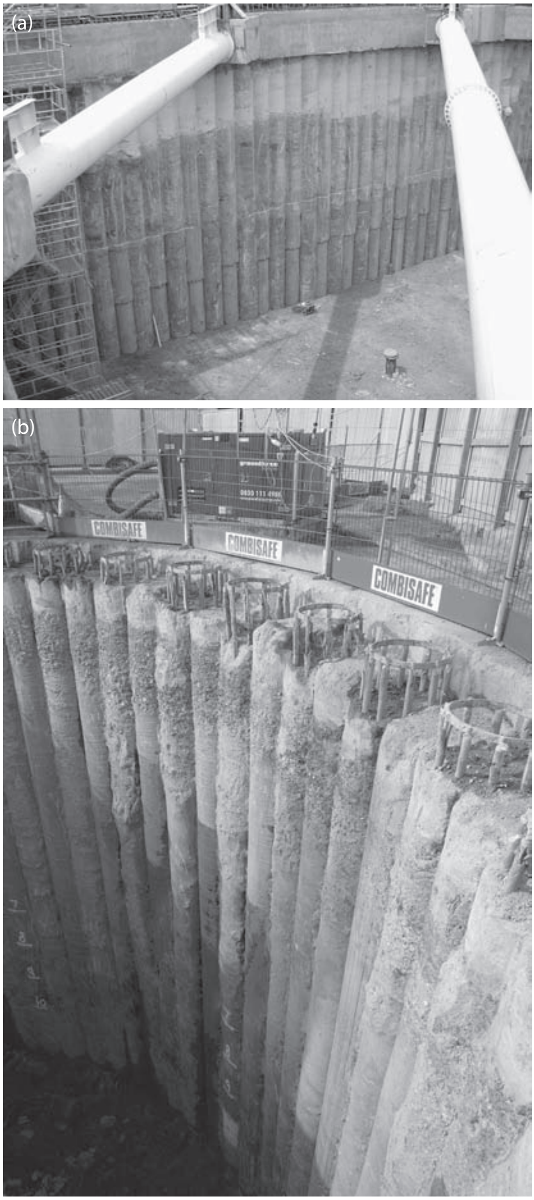


Figure 12.14 Secant pile wall. (a) Secant pile wall exposed by excavation. (Courtesy of Balfour Beatty Ground Engineering, Basingstoke, U.K.) Bored pile wall is capped by a concrete beam and supported by tubular steel props. (b) Secant pile wall exposed, showing unreinforced female piles and reinforced male piles. (Courtesy of Bachy Soletanche, Burscough, U.K.)

Secant pile walls are sometimes categorized based on the nature of the female piles (the males piles are almost always reinforced structural concrete).

1. Hard–soft walls. The female piles (“soft” piles) are constructed from unreinforced cement–bentonite slurry. These walls are essentially temporary works, because the female piles do not have the long-term durability to form part of the permanent works. These walls are sometimes included in permanent works by the addition of a structural concrete liner on the inner face once exposed by excavation.
2. Hard–firm walls. The female piles (“firm” piles) are constructed from concrete to allow them to be incorporated into the permanent works. The concrete used in the female piles is significantly weaker than in the male piles to avoid constructability problems and make overcutting of female piles easier.
3. Hard–hard walls. The female piles are still unreinforced but are formed from structural concrete of equal strength to the male piles. Special piling rigs are needed to perform the overcutting of the male piles into the high-strength female piles.

For pile diameters in the range of 600–1200 mm, typical overcut (the distance the male piles cut into the female piles) will be in the range of 100–250 mm, depending on ground conditions and the design requirements of the wall. Construction sequence and programming is important for secant pile walls. The female piles are installed first, leaving gaps to be filled in by male piles. However, if the female piles are left too long before the male piles are cut into them, the concrete in the female piles will have developed high strengths, which will create practical problems for male pile installation. Excessive concrete strength in female piles can lead to verticality and alignment problems with male piles. It is possible for male piles to pop out of alignment at depth, resulting in groundwater flow paths through the wall. Therefore, male pile installation should be programmed to be carried out at an appropriate time (based on the concrete mix design) after the female piles are installed.

Secant pile walls can be installed to depths of up to 30 m if a suitable boring plant is available. It is important to understand that the practical depth over which an effective secant pile wall can be constructed is the depth over which pile secanting can be guaranteed. This will be determined by the piling tolerances achieved during the works and not merely on the depth that can be achieved by the piling rig. Reinforced concrete guide walls (similar to those used in diaphragm walling) are typically installed at ground level as an aid to achieving good positional and verticality tolerances.

12.8.2 Contiguous pile walls

In this application, cast in situ concrete piles (typically of 600–1200 mm diameter) are installed at spacings such that the pile casings can be installed without interfering with adjacent piles or so that the auger used to form the pile is not damaged by cutting adjacent piles. A small gap (on the order of 75–100 mm) remains between the piles (Figure 12.13b). These walls are not watertight in their conventional form, and their primary use is as an earth-retaining structure that may be incorporated into the permanent works. However, the application of grouting methods to seal the gaps can allow them to act as groundwater cutoff barriers.

12.9 GROUT BARRIERS

12.9.1 Principles of grouting

Grouting encompasses a range of techniques used in ground engineering to modify the properties of soils and rocks by the controlled injection of special fluids (grout), where they set or harden and modify the ground properties. In the context of this chapter, we are primarily interested in grouting methods that reduce the permeability of soils and rocks, although many of the methods also increase the soil and rock strength.

Rawlings et al. (2000) define grouting in the following terms:

> Grouting in ground engineering can be defined as the process of controlled injection of material, usually in a temporary fluid phase, into soil or rock, where it stiffens to improve the physical characteristics of the ground for geotechnical engineering reasons.
>
> In such a situation, grouting is a process in which the remote placement of a pumpable material in the ground is indirectly controlled by adjusting its rheological characteristics and manipulating the placement parameters (pressure, volume, and flow rate).

There are seven principal types of grouting techniques:

1. *Permeation grouting*. Filling or partially filling of permeable pores within a soil by grout injection without disturbing the structure of the soil.
2. *Rock grouting*. Filling or partially filling of fissures, fractures, or joints in a rock mass by grout injection without creating new fractures or opening existing fractures.
3. *Jet grouting*. Disruption of existing soil structure by water/air/grout jets and in situ mixing with and replacement by injected grout.

4. *Soil mixing.* Disruption of existing soil structure by mechanical tools (e.g., augers or cutters) and in situ mixing with and replacement by injected grout.
5. *Hydrofracture grouting.* Deliberate fracturing of the ground (soil or rock) using grout under pressure. Typically used to inject grout into otherwise inaccessible voids to reduce permeability or increase strength.
6. *Compaction grouting.* Injection of stiff mortar or paste-like grout into the ground to displace and compact the soil in situ.
7. *Compensation grouting.* Use of compaction, permeation, or hydrofracture grouting in a controlled responsive manner as an intervention between an existing structure and an engineering operation. A typical example is compensation grouting associated with tunneling works, where it is applied to minimize tunneling-induced movements on existing structures (Mair et al. 1995).

Permeation, rock, and jet grouting are the main types of grouting used to form groundwater cutoff barriers to exclude water from civil engineering excavations. Each of these techniques is described in the subsequent sections. In addition, soil mixing can be used to form mix-in-place barriers, which are sometimes used for groundwater exclusion. Mix-in-place barriers are described in Section 12.10.

Further background information on grouting methods is given in the work of Bell (1994), Rawlings et al. (2000), and BS EN 12715:2000.

12.9.2 Types of grout

Grouts used for groundwater exclusion purposes on civil engineering projects fall into two main classes: (1) suspensions and (2) solutions.

Suspension grouts comprise solids suspended in water without being dissolved. True suspension grouts contain particles that are large enough to settle under gravity, with the associated problems of maintaining good grout quality during mixing and handling. Colloidal grouts are those in which the particles are so fine such that there is no settlement under gravity and particles are kept in suspension by Brownian motion. The most common form of suspension grouts are cement-based grouts, which consist of mixtures of cement and water, and sometimes with additives such as bentonite or PFA. The finite size of the solid particles in suspension grouts limits their ability to permeate or penetrate further into small pores or fissures in soils and rocks.

Solution grouts do not contain solid particles and generally “gel” (solidify) on setting. The most common form of solution grouts are the so-called “chemical grouts”; these materials are most commonly silicate compounds or resins. The lack of particles in solution grouts means that they typically can permeate or penetrate further into small pores or fissures in soils and rocks compared to typical cement-based suspension grouts.

Table 12.1 Indicative grouts for different types of ground

Type of ground	*Soil/Rock properties*	*Permeation grouting*	*Fissure or contact grouting*
Granular soil	Gravel, coarse sand, and sandy gravel $k > 5 \times 10^{-3}$ m/s	Pure cement suspensions Cement-based suspensions	
	Sand $5 \times 10^{-5} < k < 5 \times 10^{-3}$ m/s	Microfine cement suspensions Solutions	
	Medium to fine sand $5 \times 10^{-6} < k < 1 \times 10^{-4}$ m/s	Microfine cement suspensions Solutions Special chemicals	
Fissured rock	Faults, cracks, karst e > 100 mm		Cement-based mortars Cement-based suspensions (clay filler)
	Cracks, fissures 0.1 mm < e < 100 mm		Cement-based suspensions Microfine cement suspensions
	Microfissures e < 0.1 mm		Microfine cement suspensions Silicate gels Special chemicals

Source: BS EN 12715:2000, *Execution of Special Geotechnical Works—Grouting*, British Standards Institution, London.

Note: e = fissure width.

Table 12.1 summarizes the range of typical ground types that can be treated by various grouts. Characteristics of the grout types most commonly used in groundwater exclusion applications are given in the following sections.

12.9.3 Cement-based grouts

Cement-based suspension grouts are used widely around the world. This reflects the wide availability of the raw materials and the relative simplicity and low cost of grout preparation.

In their simplest form, cement-based grouts are suspensions of cement in water, with the water–cement ratio manipulated to provide the required rheological properties. The stability (resistance to sedimentation of particles) of a grout for a given water–cement ratio may be improved by the addition of clay, bentonite, or chemical additives. For filling large voids (old mine workings, karst voids), a filler such as PFA may be added as a substitute for cement and to reduce the cost per unit volume of grout. Accelerators may be added to decrease setting time.

Microfine cement is cement that has been ground to smaller size so that all particles are less than a specified size, typically 20 μm. Grouts using microfine cement therefore contain smaller particles and have the potential for penetrating/permeating smaller openings (soil pores/rock fissures). Microfine cement is significantly more expensive than conventional OPC, and the grout formed from it will be correspondingly expensive. It is typically only used to grout fine openings in soil or rocks that cannot be penetrated by OPC-based grouts.

The penetrability of cement-based grouts can also be improved by decreasing the grout viscosity and gel strength (using additives to reduce viscosity by flocculation). Alternatively, penetrability can be aided by increasing the resistance to filtering effects by the addition of activators (dispersing agents).

12.9.4 Chemical grouts

The most common types of chemical grouts are silicate-based grouts and resins.

Silicate grouts are based on sodium silicate in liquid form, to which a hardening agent is added. The viscosity of the mix changes over time, reaching a solid state as a gel. By varying the type and concentration of chemical components in the grout, a wide range of properties can be obtained. There are two principal types of gel:

1. Soft gels (water-tightening gels)
2. Hard gels (strengthening gels)

Soft gels have a low concentration of silicate, with gelling typically obtained by adding mineral reagents. The low viscosity (similar to that of water) means that they can be injected into fine sands where they have a "water-tightening" effect, reducing permeability. These grouts have a short design life (around 6 months to 2 years) and are therefore only suitable for temporary works of relatively short duration. The permeability of treated ground may increase over time due to the degradation of the grout by various processes including washing out and erosion.

Hard gels have a higher silicate concentration and organic reagent content and produce a stronger gelled grout. Hard gels are more viscous than soft gels (and, therefore, are less effective at penetrating small openings than soft gels). The design process may need to address the two conflicting issues of grout strength and durability versus penetrability of the grout into the target ground.

The development of colloidal silica grouts has allowed very fine grained sands to be successfully grouted. These grouts consist of colloidal-sized particles of silica that are coagulated with a second component of a simple salt solution. The advantage of colloidal silica grouts is that they are permanent and do not suffer the durability problems of silicate grouts.

Resin grouts are based on solutions of organic products, either in water or in nonaqueous solvents, which can form a gel under given conditions. Resin grouts can potentially have very low viscosity (and, hence, can penetrate very small soil and rock openings) and setting times that can be controlled between a few seconds to several hours (depending on the grout components and reagents). Resin grouts are typically much more expensive than other types of chemical grouts. They are most typically used where durable grouts are required and no other grout types are feasible.

Resin types used for permeability reduction and water exclusion purposes include acrylic, polyurethane, and phenolic grouts. Most resins are of significant toxicity and should only be used by experienced designers and contractors, with suitable health protection and control measures in place.

12.9.5 Design of grout barriers

It should be recognized that the successful installation of a groundwater cutoff barrier by grouting methods is a complex process and will be influenced by many factors, including ground conditions, the fabric and structure of the ground, grout properties (rheology), and the method of grout delivery. The eminent civil engineer Sir Harold Harding (one of the pioneers of modern grouting techniques in British construction practice) once said that one of the challenges of grouting was, "Manipulating it in the ground beyond vision or arm's length" (Harding 1947). It is certainly true that injecting grout into the ground is one thing, but delivering it accurately to the desired location and achieving the required permeability reduction may be quite another challenge.

A grouting solution for a given site should be developed on a bespoke basis based on a thorough ground investigation, and the expert advice of experienced grouting designers should be obtained. On projects of any significant scale or where success of grouting is critical, a trial section of grouting is often carried out in advance of the main cutoff works to allow the grouting design to be finalized.

A key element of the design of a grouted groundwater barrier is the selection of the grout type and properties. The choice of grout is influenced primarily by the permeability of the ground, particularly the likely size range of the interconnected voids in soil or interconnected fissures in rock. In soils, an empirical parameter known as "groutability ratio" (defined as ratio between the particle sizes representing 15% of the ground and 85% of the grout particles) can be used to assess the penetrability of particulate grouts. In rocks, maximum grout particle size to fissure width is considered, with a ratio of 3 being commonly used. Table 12.1 summarizes the range of typical ground types that can be treated by various grouts.

Various grouting techniques have the potential for generating significant volumes of solid and slurry spoil, which must be disposed of appropriately and in accordance with the relevant regulations. In many cases, the spoil will contain greater or lesser quantities of grout; thus, the practical and legal aspects of disposal of cement-based or chemical grouts must be considered.

In most cases, grouted groundwater barriers are effectively permanent and will remain in place long after construction works are finished. It may be necessary to assess the long-term impact of the change to groundwater flow patterns (see Section 15.6). On rare occasions, where the groundwater chemistry has the potential for interacting with the grout barrier, the risk of leaching of polluting chemicals from the grout into the groundwater may need to be assessed. This is especially relevant for PFA-based grouts that can leach heavy metals and are now becoming increasingly less favored for this reason.

In addition to the usual health and safety issues on civil engineering sites, there are some risks particularly associated with the use of grouts. Cement-based grouts can create dust problems during mixing, and there is a risk of "cement burns" if liquid grout is allowed to stay in prolonged contact with skin. Chemical grouts are often harmful to the skin and may give off potentially toxic or irritant fumes. Appropriate health and safety measures should be put in place during grouting operations.

12.9.6 Permeation grouting

Permeation grouting involves the injection of liquid grout, typically via drill holes, at a steady injection pressure, without disturbing the soil structure. It is used in soils to fill or partially fill the pore spaces, displacing the water and air in those spaces. The grout sets or gels in the pore spaces, reducing the potential for intergranular groundwater flow, thereby reducing permeability (Figure 12.15).

In groundwater control applications, permeation grouting is commonly carried out from lines or grids of closely spaced drill holes. The intention is that the grout permeates radially from each drill hole and merges with the grouted zones around neighboring drill holes. This way, a more or less continuous "grout curtain" of lower mass permeability than the surrounding ground can be formed.

The sequence for permeation grouting is typically injection into each drill hole of a finite predetermined volume of grout, in a sequence of primary, secondary, and possibly tertiary holes. The intention is for the primary injections to create grout zones that overlap or coalesce to some degree.

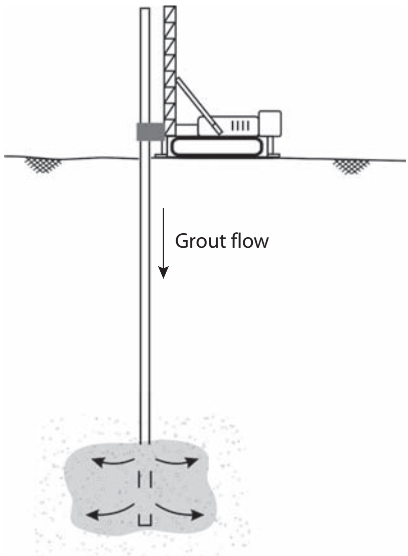


Figure 12.15 Permeation grouting. Liquid grout is injected at a steady pressure, without disturbing soil structure, to fill or partially fill the soil pore spaces.

Final closure between the primary injections is achieved by injection into the secondary and tertiary holes. Typical final spacing between grouting drill holes varies from 0.8 to 1.3 m in fine sands up to 2–4 m in more permeable gravels.

During grouting, the effectiveness of the injection program in reducing permeability, and hence, the need for additional phases of grout injection is normally investigated by a program of "water testing." A water test is basically an in situ permeability test (see Section 6.7) that is carried out within the grouted zone. Where water tests are used as a part of the design or validation process for a grouting program, it is essential that a sufficient number of tests are carried out to ensure that test results are not skewed by local variations in natural ground conditions.

In highly permeable materials (e.g., gravels), grout injection may be directly from the end of a cased borehole (Figure 12.15). More sophisticated grouting programs may use the tube à manchette technique (TAM grouting). TAM pipes are specialist grouting tubes, typically 25–50 mm in diameter, installed in drill holes and permanently secured into place by filling the annulus around them with weak grout (sleeve grout). The TAM pipes contain grout ports at regular vertical intervals (commonly between 250 and 500 mm) protected by external rubber seals (Figure 12.16). Using a special double packer lowered down inside the TAM pipe, each level of grout ports can be isolated, allowing grout injection at a specific level, and

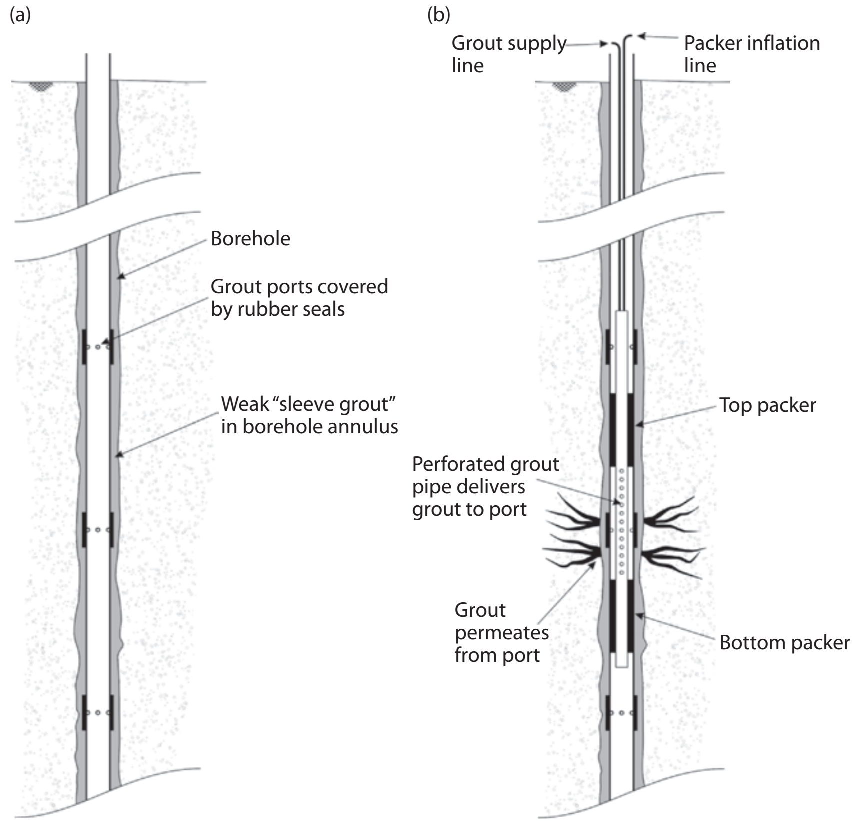


Figure 12.16 Grouting using TAM. (a) TAM pipe installed in a borehole. (b) Grout injected via double packer inside a TAM pipe.

injection pressures and volumes can be monitored and controlled at each level. When grout is injected, the grout pressure opens the rubber seal covering the grout ports, breaks the sleeve grout, and allows grout to pass into the soil. The TAM method allows controlled grouting, because the level of grout injection can be controlled, and repeated injections can be made from any grouting port.

Appropriate selection of grout characteristics is fundamental to successful permeation grouting. For example, when cement-based suspension grouts are used, flow blockages may occur in the soil pores due to filtration of grout particles in the flow paths. This may prevent grout flowing the necessary distance from the injection drill hole. The viscosity of the grout, both on injection and later as the grout begins to gel, will also restrict grout flow. The applicability of grouts is summarized in Table 12.1. In general, the effective use of low-pressure permeation grouting using conventional cement-based grouts is limited to gravels with modest sand contents. The use of microfine cement grouts and chemical grouts may allow permeation grouting to be effective in medium and possibly fine sands.

The reduction in permeability of treated ground that can be achieved by permeation grouting will be affected by the grout design and placement methods, but the achievable effects will be dominated by the nature of the soil. In gravels and medium to coarse sands, a treated permeability on the order of 1×10^{-6} m/s is achievable with good practice.

The grout used is normally designed to penetrate the finest groutable soil pores based on the objectives of the grouting program. Sometimes, different grout types are used in a grouting program. Cheaper cement-based grouts may be used for a sequence of initial grouting to fill the larger void spaces, followed by a later sequence of grouting with more expensive chemical grouts to fill as many of the remaining smaller void spaces as possible.

In general, well-controlled permeation grouting will produce only modest quantities of spoil (associated with the drilling of injection holes). Low-pressure permeation grouting should not cause problems of ground heave.

Further information on permeation grouting is given in the work of Rawlings et al. (2000) and BS EN 12715:2000.

12.9.7 Rock grouting

Rock grouting is analogous to permeation grouting but is applied specifically with the objective of injecting grout to penetrate and fill or partially fill fissures, fractures, joints, and other voids within a rock mass. The grout gels or sets, blocking water flow pathways through the rock, thereby reducing mass permeability (Figure 12.17).

Rock grouting will typically strengthen rock formations, because the presence of hardened grout will restrict the potential for movement along fissures or joints in the rock mass. In civil engineering applications, rock grouting is used to reduce groundwater inflows in excavations into rocks for shaft sinking, tunnel construction, and dry docks. Rock grouting has also been used to provide low-permeability basal seals (Figure 12.1d) to reduce inflow into the base of excavations as described by Davis and Horswill (2002).

Rock grouting is typically carried out in stages, from small-diameter (typically 50 mm) drill holes. In descending-stage grouting, the drill hole is grouted in stages from top to bottom, at intervals during drilling of the hole. In the ascending stage, grouting of the drill hole is first completed to full depth and is then grouted in stages from bottom to top. Typically, for each stage, the relevant part of the borehole is isolated by packer, allowing grout injection to be targeted at specific zones.

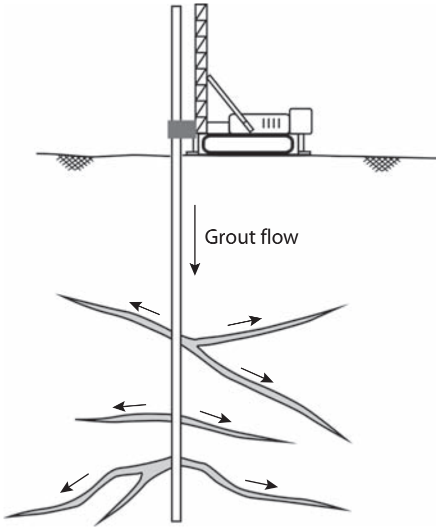


Figure 12.17 Rock grouting. Liquid grout is injected at a steady pressure to fill or partially fill fissures within the rock.

Spacing between rock grouting drill holes in a completed grouting pattern is typically in the range of 1–4 m, usually in single or multiple parallel lines, depending on the size of discontinuities to be penetrated and the target permeability of the treated ground. In rock grouting, the drill holes are typically drilled in a "split-spacing" sequence, in which additional holes are drilled and grouted between previously grouted holes. Primary holes would be drilled at a wide spacing (perhaps 5–6 m) between holes. These holes would be grouted before secondary holes would be drilled and grouted in the intervening spaces, halving the effective grout hole spacing. Tertiary holes may then be drilled and grouted between the primary and secondary holes, and so on. At each stage, the grout take (volume of grout injected before a predetermined "refusal" pressure is reached) for each hole would be recorded, and a series of "water tests" (in situ permeability tests) would be carried out. As later stages of holes are grouted, grout takes should reduce, and results of water tests should indicate reduction in permeability. These data are used to determine the need for additional injections.

The primary and secondary injections are intended to fill the larger fissures and discontinuities in the rock. Tertiary and later stages of grouting should grout the finer discontinuities. Typically, the grouting is carried out with a number of grout mixes, initially using "thin" grouts at high water–cement ratios with fixed volumes. If the initial grout volume is injected without resistance, then the next thicker grout volume is injected until a pressure buildup is seen. Later stages of grouting may be carried out with more penetrating grouts such as microfine cements or, occasionally, chemical grouts.

In relation to the planning and monitoring of grouting operations, the permeability of rock masses is sometimes referred to in lugeon units. Lugeon units describe the potential for fissured rock to accept water as assessed by a packer test (see Section 6.7.8) under specific conditions. One lugeon is defined as a water acceptance of 1 L/min through a 1-m length of a 76-mm-diameter borehole under an applied head of 100 m above groundwater level for a period of 10 min. Lugeons can be related to conventional SI units, as 1 lugeon is approximately equal to 1×10^{-7} m/s.

Most rock grouting is carried out using cement-based grouts. Conventional cement grouts are effective in sealing rock openings of 0.5 mm or larger and can achieve a minimum permeability of around 5 lugeons (approximately 5×10^{-7} m/s). With the use of microfine cement or chemical grouts, it may be possible to achieve a permeability of the grouted mass of around 1 lugeon (approximately 1×10^{-7} m/s) or less.

Further information on rock grouting is given in the work of Houlsby (1990), Rawlings et al. (2000), and BS EN 12715:2000.

12.9.8 Jet grouting

Jet grouting involves the erosion of the natural soil structure by high-pressure water or grout jets, flushing away a significant proportion of the soil particles, which are replaced by grout to produce a mixture of grout and soil.

The process normally involves a jetting monitor (from which the jets emit radially) being raised out of the ground on a slowly rotating specialist drill string or jetting pipe (Figure 12.18). Correctly executed, this will produce a column of treated material (a “jet-grouted” column) that is of low permeability and potentially of significantly greater strength than the untreated soil. Continuous barriers are achieved by the installation of overlapping jet-grouted columns. Jet grouting has also been used to form basal plugs (Figure 12.1d) within excavations, as described by Newman et al. (1994). Because of the potential strength of jet-grouted material, it can play a role in temporary ground support during excavation. A wide range of soils can be treated, both granular and cohesive, as well as weak and weathered rock. Permeabilities of 10^{-7} to 10^{-9} m/s have been achieved in jet-grouted barriers.

A jet-grouted column is formed by drilling in the jetting pipe (typically 100–200 mm in diameter) to the maximum depth of the column (in difficult ground conditions, a predrilled hole may be used). One or more high-pressure radial jets of fluid are then emitted from the monitor at the base of the jetting pipe (Figure 12.19) while the pipe is rotated slowly. The jetting pipe is then lifted slowly while rotating to create the column of mixed soil and cement.

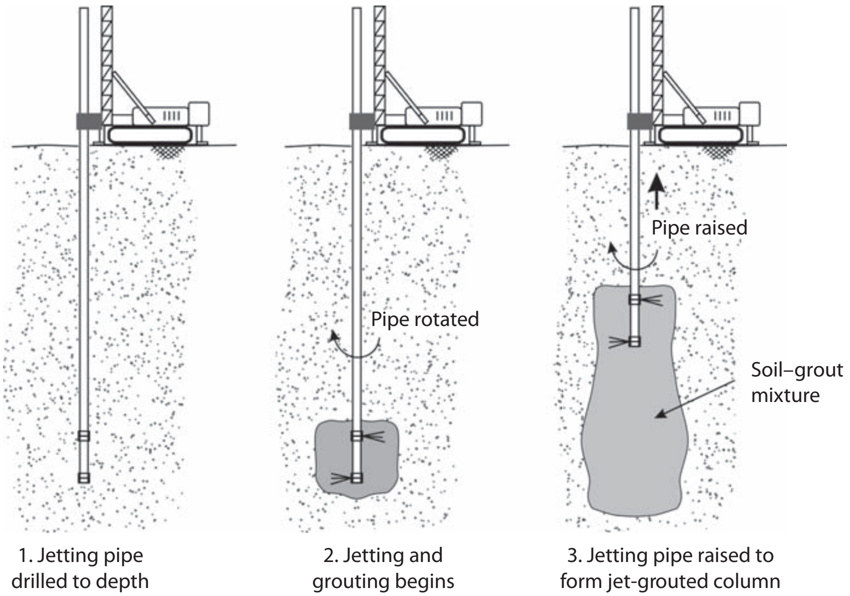
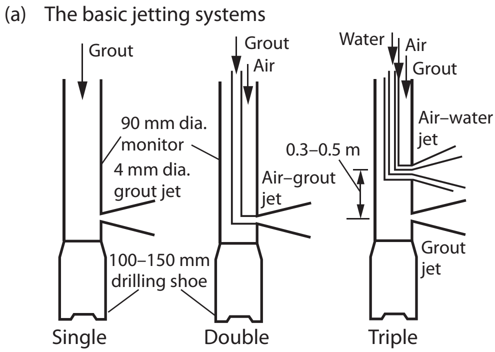


Figure 12.18 Jet grouting. High-pressure water or grout jets are used to form a column of disturbed soil around the jetting pipe, flushing away a significant proportion of the soil particles, which are replaced by grout to form a column of soil–grout mixture.

Jet grouting is normally carried out using cement-based grouts, either cement only or cement with bentonite or other fillers.

Three types of jet grouting systems are available (Figure 12.19):

1. *Single jet.* A high-pressure grout jet (typically at 30–60 MPa) is used to cut the column, eroding and replacing the soil. Excess soil and grout slurry (spoil return) is forced up the annulus around the jetting pipe to the surface. The primary mechanism with the single-jet method is injection and mixing of grout, with only limited replacement of soil.
2. *Double jet.* A grout jet is shrouded in compressed air (typical pressure of 0.2–1.5 MPa). The purpose of the air shroud is to improve the cutting action of the jet, increase the energy of the jet, and, hence, allow a greater volume of ground to be treated by a single-column insertion. The double-jet method typically involves greater soil replacement (and, hence, spoil volumes) than the single-jet method.
3. *Triple jet.* A water jet that is shrouded in air is used to erode the soil, with a separate nozzle used for grout injection (typical pressures: air 0.5–1.5 MPa; water 4 MPa; grout 0.5–3 MPa). The triple-jet method typically involves greater soil replacement (and, hence, spoil volumes) than the single-jet method.



(b)

Figure 12.19 Jet grouting equipment. (a) Types of jet grouting systems. (From page 78 of Woodward, J., *An Introduction to Geotechnical Processes*, Spon Press, London, 2005. With permission.) (b) Jet grouting rig. The radial jets at the base of the drill string are shown operating above ground level. (Courtesy of Keller Geotechnique, Wetherby, U.K.)

The diameter of ground treated by a single-column insertion depends on the method used, soil type, grout characteristics, injection pressures, and lift and rotation speeds. Typical column diameters of treated ground for a single-jet-grouted column are 0.5–1.2 m for the single-jet methods and up to 3.0 m for the double- or triple-jet method.

Jet grouting operations are complex and must be carefully controlled. Three potential problems or challenges with jet grouting are

1. Ground heave if grout injection and spoil removal are not carefully controlled
2. Hydrofracturing of the ground as a result of spoil returns becoming blocked
3. Generation of large volumes of slurried spoil (of the order of 50 m^3 of spoil per rig per shift)

Further information on jet grouting is given in the work of Essler (1995), Lunardi (1997), and BS EN 12716:2001.

12.10 MIX-IN-PLACE BARRIERS

Mix-in-place barriers are low-permeability barriers formed in situ by “soil mixing,” where the mechanical action of an auger or cutter, in combination with grout injection into the mixing zone, creates a zone or block of treated material. When used appropriately, very low-permeability barriers can be formed, with little or no spoil generated. Two main types of soil mixing have developed. In dry-soil mixing, a dry powder (often cement or lime–cement) is injected into the in situ mixing zone. In contrast, wet-soil mixing involves the injection of a liquid additive (typically a cement-based grout) into the mixing zone. For civil engineering applications, to create barriers to groundwater flow, wet-soil mixing is the most common method used. The following section deals exclusively with wet-soil mixing.

When mix-in-place barriers were first developed in the 1990s, they were based on the use of modified augers as the mixing tool for forming a vertical cutoff barrier formed from overlapping columns of treated material. In this application, the geometry and installation sequence is very similar to that used for secant pile walls (see Section 12.8). Single-axis installation rigs use a single modified auger to form a column of treated material. Grout is injected down the stem of the auger during insertion and removal, with the auger often being “worked up and down” in the bore to ensure thorough mixing. Soil mixing is often carried out with modified augers with multiple grout injection points, mixing paddles, and gaps in the auger flights to promote mixing. Multiaxis rigs that install two, three, or more interlocking columns of treated ground in one cycle are available. Nominal column diameters are in the range of 600–2500 mm. Depths on the order of 40 m are possible with land-based equipment.

Since the early 2000s, an alternative construction method for mix-in-place walls has emerged in the form of cutter soil mixing (CSM) equipment, derived from hydromill cutters used for diaphragm wall construction (Schöpf 2004). In contrast to auger-based soil mixing, where the cutting tool rotates on a vertical axis to form a column, CSM rigs use multiple cutting wheels rotating on a horizontal axis to form panels. As the cutters disturb the soil, grout is injected to form the mix-in-place material. Wall construction is achieved in an analogous way to diaphragm walls, whereby discrete panels are formed, each cutting into its neighbor to achieve an overlap. CSM units are typically mounted on a crawler-mounted base (Figure 12.20). This technique can be used to construct walls to depths of 60 m and thicknesses of up to 1.2 m.

Vertical mix-in-place barriers can also be installed using modified trenching machines, as described by Hardwick (2010). Special trenchers, similar in principle to those used to install horizontal wellpoints (see Section 11.3), use a rapidly moving chain cutter/mixer to break up the ground and form a narrow slot of disturbed material while the grout is injected (Figure 12.21). The direction of the cutter/mixer chain is periodically reversed to ensure adequate mixing. Trenchers are available to install barriers to depths of 6–7 m.

Figure 12.20 CSM unit. (Courtesy of Golder Associates, Toronto, ON, Canada.)

Figure 12.21 Mix-in-place barrier installed by trencher. (Courtesy of Bachy Soletanche, Burscough, U.K.)

In appropriate conditions, mix-in-place methods have the potential to produce barriers with permeability in the range of 10^{-7} to 10^{-9} m/s. The treated material is typically of higher strength than the natural ground and may potentially, with appropriate design, play some role in the temporary works structural support of an excavation.

Further information on mix-in-place barriers is given in the work of BS EN 14679:2005, Greenwood (1989), and Blackwell (1994).

12.11 ARTIFICIAL GROUND FREEZING

Artificial ground freezing involves the installation of a series of closely spaced, small-diameter boreholes (known as freezeholes) typically to form a ring around an excavation. A very low temperature refrigerant is then circulated through the freezeholes. This chills the ground around each freezehole, freezing the water contained within the soil pores or rock fissures and causing a cylinder of frozen ground to form around each freezehole. As the circulation of the refrigerant continues, the cylinders of frozen ground will slowly increase in diameter. The objective is for the cylinders to eventually intersect and coalesce to form a continuous "freezewall" around the excavation (Figure 12.22). The formation of a complete freezewall is termed "closure."

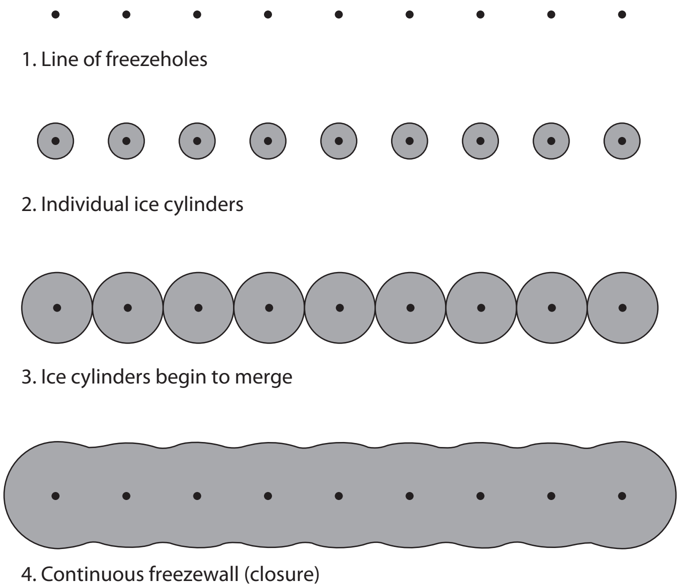


Figure 12.22 Development of a freezewall.

A suitably designed freezewall will be of very low permeability and can be very effective in excluding groundwater from an excavation or tunnel. Frozen ground is also very strong, and a freezewall can form part of temporary works structural support for an excavation.

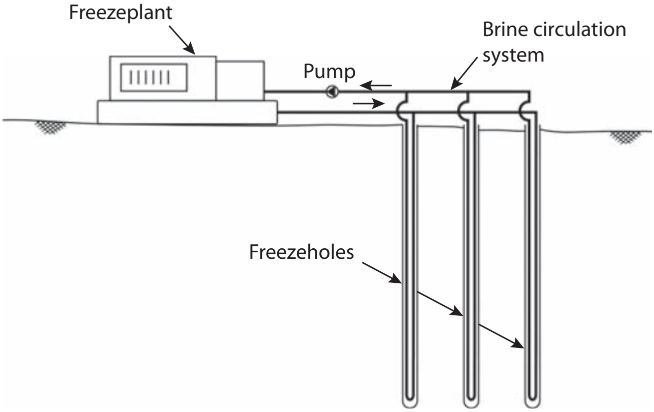
Unlike most other techniques for groundwater exclusion, artificial ground freezing is truly temporary. A freezewall can only be maintained by the continued circulation of refrigerant, although once the freezewall has been established, refrigerant flows can normally be reduced relative to levels at the start of freezing. Therefore, upon the completion of a project, refrigeration is normally discontinued, and the freezewall will normally thaw slowly (probably over a period of months), leaving behind no permanent barrier to groundwater flow. This may be a benefit in areas where there are concerns over long-term impacts on groundwater flow (see Section 15.6).

Two principal types of systems used for artificial ground freezing: (1) the circulation of chilled brine in a closed system and (2) the use of liquid nitrogen (LN).

12.11.1 Brine circulation systems

This configuration uses a closed circuit of low-temperature brine (typically calcium chloride) circulated through the freezeholes (Figure 12.23).

(a)



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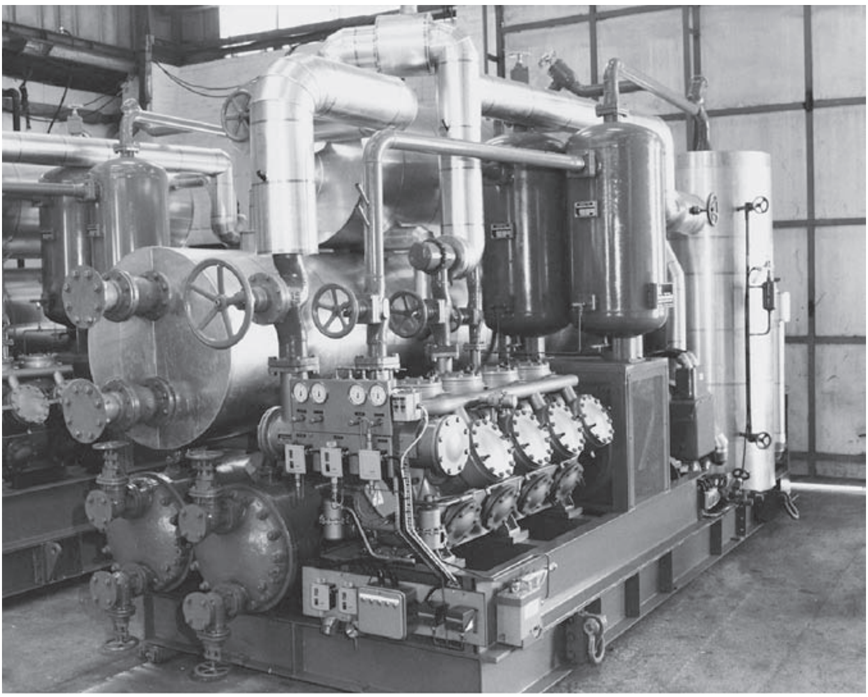

Figure 12.23 Artificial ground freezing using brine circulation. (a) Schematic view of brine circulation system. (b) Portable freezeplant. This freezeplant is driven by a 180-kW electric motor. The output is 166 320 kcal/h when evaporating at −37.5°C. (Courtesy of the British Drilling and Freezing Company Limited, Nottingham, U.K.)

The brine is circulated to a portable freezeplant (effectively a large refrigerator) powered by electric or diesel prime movers, which dissipates the extracted heat to the atmosphere via cooling towers or evaporative condensers.

Brine is typically circulated at temperatures of between –25°C and –35°C. At these circulation temperatures, the establishment of a complete freezewall (time for closure) is relatively gradual and may take several weeks.

12.11.2 LN systems

In contrast to brine systems, this configuration of artificial ground freezing does not rely on mechanical refrigeration plant. Instead, a reservoir of LN is delivered to the site and held in a large insulated storage vessel. LN is extremely cold (–196°C) in liquid form. When passing through the freezeholes, the LN evaporates, thereby absorbing the latent heat of vaporization and creating an intense refrigerant effect that rapidly chills the ground (Figure 12.24). The resulting nitrogen gas is vented to the atmosphere, often causing plumes of water vapor when the cold gas causes condensation when it meets warmer atmospheric air.

LN systems operate in a much lower temperature than brine systems. The temperature in freezeholes is typically between –100°C and –196°C. Establishment of a complete freezewall is much more rapid for LN systems compared to brine systems, with a complete freeze typically being established between 3 to 10 days.

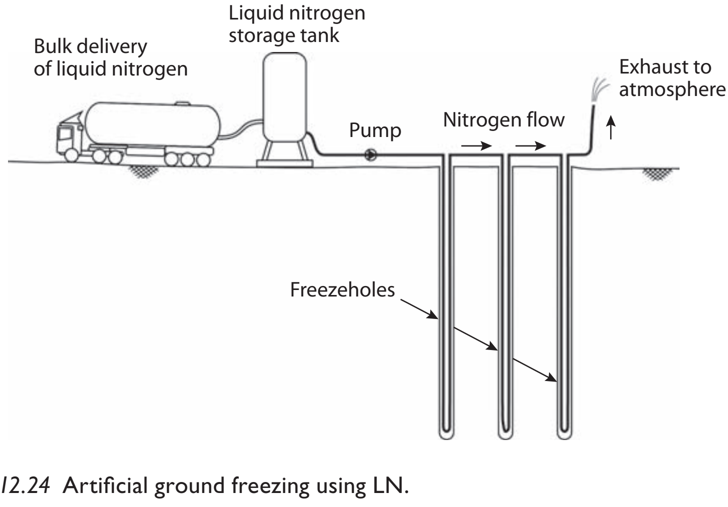

Figure 12.24 Artificial ground freezing using LN.

12.11.3 Typical applications for artificial ground freezing

There are two typical situations when artificial ground freezing is used:

1. *Planned use.* This is when ground investigations have identified the potential for difficult excavation conditions, for example, very unstable water-bearing strata, very deep excavations, and so on. In such cases, artificial ground freezing is planned for from the start, typically with the twin objectives of preventing groundwater inflow and temporarily increasing the strength of the soil or rock around the excavation. In these cases, freezing can be either by brine or LN systems, depending on ground conditions and the time available to establish the freezewall (LN being used in situations when the freezewall must be established rapidly).
2. *Emergency (recovery) use.* There have been numerous occasions when artificial ground freezing has been used as part of recovery operations following collapse and large-scale instability or inundation of an excavation or tunnel. Following such events, there is normally a need to control groundwater and stabilize the ground to allow remedial works to be carried out. Because of the need for rapid establishment of freeze and the difficulties of stabilizing the disturbed ground around a collapse, LN is typically the refrigerant of choice in these applications. Clarke and Mackenzie (1994) and Brown (2004) describe two case studies where artificial ground freezing was used as part of recovery works for tunneling projects that had encountered difficulties.

Typical configurations of artificial ground freezing installations include vertical freezewalls around shafts or excavations (Figure 12.25) or horizontal freezes to provide support around tunnels and other underground spaces.

The freezeholes are typically of relatively small diameter (100–150 mm). Typical design spacing between freezehole centers is of the order of 1 m. It is essential that drilling is carried out within accurate tolerances of setting out, alignment, and straightness so that two adjacent freezeholes do not diverge at depth. If this occurs, the time for the freezewall to “close” may be longer than anticipated, or in extreme cases, closure may not be achieved in the area of the misaligned freezeholes. Oftentimes, freezeholes are accurately surveyed along their length prior to commencement of refrigerant circulation so that any areas of misaligned freezeholes can be identified and additional holes can be drilled if necessary.

The development of the freezewall is commonly monitored by means of thermocouples located in boreholes located close to the freezewall.

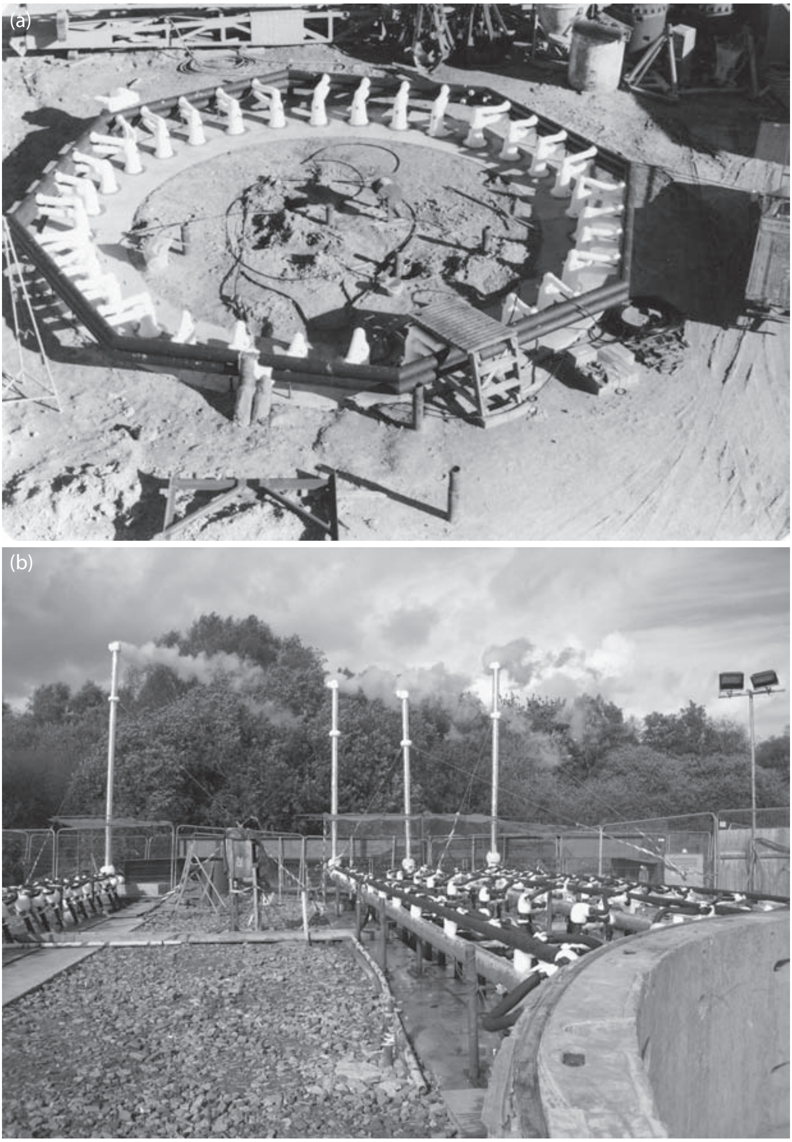

Figure 12.25 Typical layouts of artificial ground freezing systems. (a) Artificial ground freezing system (using brine as the refrigerant) prepared ready for sinking of a circular shaft. (b) Artificial ground freezing system (using LN as the refrigerant) as part of the recovery works on a tunnel project. The white plumes are water vapor caused by the low temperatures of the nitrogen gas exhausted to the atmosphere from the four towers visible in the background. (Courtesy of the British Drilling and Freezing Company Limited, Nottingham, U.K.)

12.11.4 Limitations of artificial ground freezing

The principle of artificial ground freezing is that the circulation of refrigerant in the freezeholes will chill and, ultimately, freeze the groundwater in pores and fissures. The success of the technique will be dependent on the nature of the groundwater regime.

If the soil or rock is unsaturated (e.g., above groundwater level or where "dry" cavities exist), then there may be insufficient free groundwater to allow artificial ground freezing to be effective.

Flowing groundwater can also impede successful ground freezing. Groundwater flowing across a freezewall will carry heat away, reducing the net cooling power available to establish or maintain a freezewall. This will increase the time to achieve closure of the freezewall or may even prevent a complete freezewall from being established. Typically, brine circulation systems may experience problems if groundwater flow velocities are greater than about 2 m/day. The lower refrigerant temperatures used in LN systems mean that these systems may potentially be effective in groundwater flow velocities up to 20 m/day.

Ground movements (heave and settlement) due to freeze-and-thaw effects are sometimes associated with artificial ground freezing as a result of the formation and thawing of ice lenses. Such ground movements are of particular concern in clays, silts, and organic soils.

Very cold pipework is a necessary part of artificial ground freezing systems, which presents specific health and safety risk (cold burns). Similarly, if leaks of the refrigerant (such as LN) occur, they will be risks to personnel due to, for example, asphyxiation resulting from an excess of nitrogen gas in confined spaces such as tunnels and excavations. Health and safety protocols specific to artificial ground freezing systems should be used.

Further information on artificial ground freezing is given in the work of Harris (1995).

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Chapter 13

Pumps for groundwater lowering duties

13.1 INTRODUCTION

There is a wide range of pumps and pump sets available for pumping water. However, many pumps are not suitable for temporary works dewatering installations; thus, equipment must be selected with care. The categories of pump sets appropriate to a particular groundwater lowering site requirement will depend on the technique in use.

In this chapter, the categories appropriate to the most common groundwater lowering techniques (wellpoints, deep wells, and sump pumping) are described together with some discussion of their strengths and weaknesses.

13.2 UNITS FOR WELLPOINT PUMPING

The wellpoint pump operates on the suction principle and is positioned at some elevation above the level of the installed wellpoints (see Chapter 9), generally at the level of the header main. It is required to suck the groundwater into the perforated wellpoints up the individual unperforated riser pipes and into the suction header main to the intake of the wellpoint pump set and then discharge the pumped groundwater via a discharge manifold to a disposal point. It follows, therefore, that an efficient wellpoint pump set must develop

1. Adequate vacuum to lift the groundwater from the soil at the level of the wellpoints and deliver the groundwater to the pump intake
2. Sufficient residual power to discharge the pumped water to the disposal area

All efficient wellpoint pump sets are designed to pump only clean water, with minimal suspended solids. There are three categories of pump types commonly used for wellpoint applications:

1. Double-acting piston pumps
2. Self-priming centrifugal pumps
3. Vacuum tank units

13.2.1 Double-acting piston pump

The Callans-type reciprocating double-acting piston pump (Figure 13.1) is sometimes employed on wellpoint installations in the United Kingdom, in contrast with continental Europe, where it is widely used. Its energy consumption is significantly less than that of a vacuum-assisted self-priming centrifugal pump set of comparable output. If there are suspended fine particles in the abstracted groundwater (perhaps because the sanding-in of wellpoints is inadequate), these will act as an abrasive that will cause wear of the pistons and piston liners. In time, this will lead to a reduced vacuum and associated deterioration of pumping efficiency.

Figure 13.1 Reciprocating piston pump set used for wellpointing. (Courtesy of WJ Groundwater Limited, Bushey, U.K.)

Units are available with diesel or electric prime movers. A typical 100-mm unit (which refers to the nominal size of the discharge outlet) has a power requirement of 5.5 kW and can pump up to 18 L/s at a 10–15 m head. The larger 125-mm unit (7.5 kW) can pump up to 26 L/s at the 10–15 m head. Comparison with the capacities of self-priming centrifugal units shows that piston pumps are generally suited to lower flow rate situations and that, if high flows are anticipated, a centrifugal unit may be a better choice.

These pumps are nominally self-priming, but even for a pump in good condition, priming may be slow. Accordingly, initial priming (by filling the pump and header main with clean water) should be considered. The amount of vacuum that it can generate when primed is slightly less than that generated by a comparable self-priming vacuum-assisted centrifugal pump set. As a rule of thumb, a piston pump is not appropriate if a suction lift of more than 4.5–5 m is required.

13.2.2 Vacuum-assisted self-priming centrifugal pump

The conventional vacuum-assisted self-priming centrifugal wellpoint pump set has four separate components:

1. An enclosed chamber (the float chamber) complete with an internal baffle and a float valve. This chamber serves to separate the air and water drawn into the pump. The inlet side of the float chamber is connected to the header main, and the outlet is connected to the pump unit. The volume of this chamber should be generous to ensure adequate separation of the air from the water. This is especially important when the rate of water flow from the header main is substantial. The air should be extracted by the vacuum pump so that the water fed to the eye of the pump impeller has little or no air; otherwise, there will be a risk of cavitation of the pump when operating. The purpose of the float valve is to shut off the vacuum when the water level in the float chamber reaches a predetermined level, thereby preventing carryover of droplets of water to the vacuum pump, and to reestablish vacuum when the water level in the float chamber has fallen.
2. A vacuum pump connected to the float chamber to augment significantly the small amount of vacuum generated by the centrifugal pump unit itself to suck the groundwater continuously into the float chamber and to remove air from the water.
3. A "clean water" centrifugal pump (i.e., with a minimal clearance between the impellers and the inside of the volute casing) to discharge

the water from the float chamber to the discharge main. Generous clearance between the impeller and the volute is only appropriate to its usage as a contractor's "all-purpose" pump (see Section 13.4) for dealing with solids in suspension, but the extra clearance needed for the all-purpose duty impairs efficient performance for a wellpointing duty.

4. The motor (prime mover) will generally be either diesel or electric. Its rating must be adequate to drive both the vacuum pump and the water pump to produce the expected flow with adequate allowance to cope with total head from all causes. If it is anticipated that the pump will be required for duty at a ground elevation significantly above the mean sea level and/or the ambient temperature will be above normal, a larger engine will be needed to produce the required output.

A centrifugal wellpoint pump is operated continuously at a high vacuum (i.e., low positive pressure at the eye of the impeller) and, therefore, is liable to cavitation. A cavitating pump exhibits a rattling sound similar to a great snoring noise, and the unit tends to vibrate. If the condition persists, the surfaces of the impeller will become pitted, and the bearings will be damaged. The pump shaft may be fractured.

The most commonly used size of centrifugal pump for wellpoint installations is the 150-mm water pump size with a 10-L/s vacuum pump (Figure 13.2a). Such pumps typically have water flow capacities of up to 55 L/s at a 10-m head. The 100-mm size is also used on some small wellpoint installations (capacity up to 40 L/s at a 10-m head). Larger units are available in sizes of 200, 250, and 300 mm but are not often required; it is generally preferable to use two or more 150-mm pump sets in place of larger units.

Wellpoint pumps are most commonly powered by diesel prime movers. These units are relatively noisy, and there is growing pressure for the use of "silenced" units in built-up areas. Silenced pumps have the entire pump and prime mover enclosed in a noise-reducing housing, as shown in the example of a 300-mm pump unit in Figure 13.2b. In fact, silenced units are far from silent; if noise is a major concern, then electrically powered pumps (running from a mains supply) should be considered, as these are the quietest units available.

The vacuum-assisted self-priming centrifugal pump may also be used for sump pumping of clean filtered groundwater (see Chapter 8) and for operating a bored shallow well system (see Chapter 10).

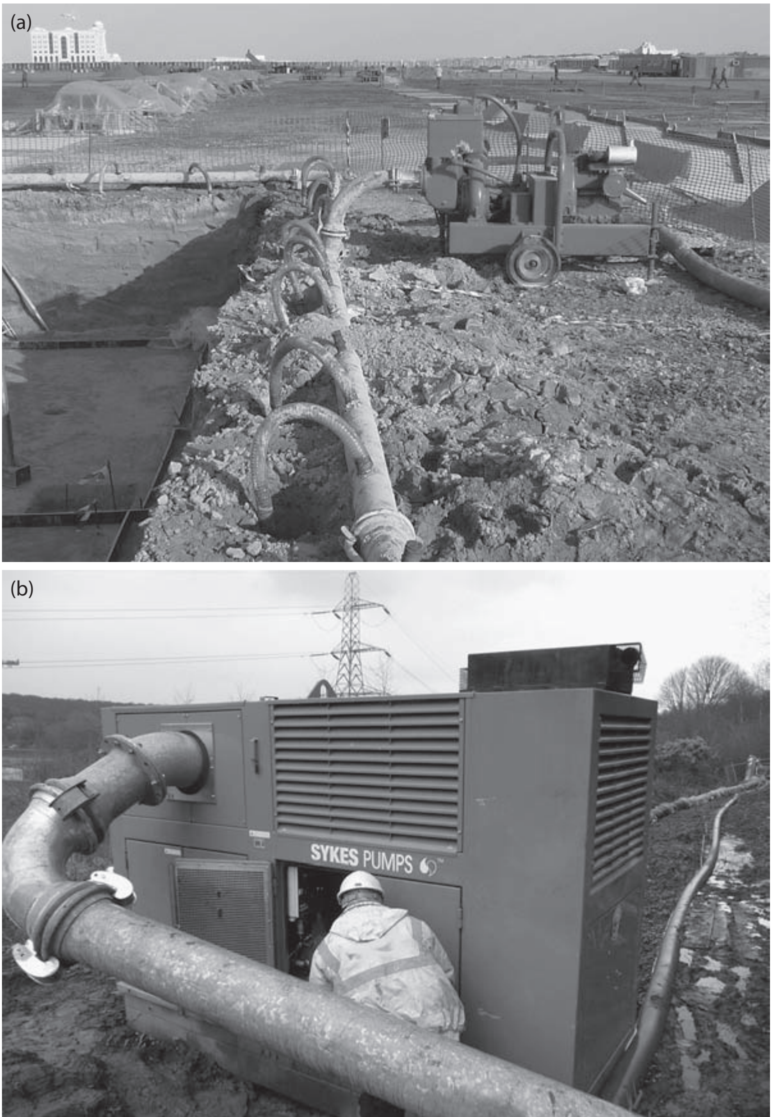


Figure 13.2 Vacuum-assisted self-priming centrifugal pump set used for wellpointing. (a) Unsilenced wellpoint pump set. (b) Silenced wellpoint pump set. (Reproduced with the permission of Andrews Sykes Group plc, Wolverhampton, U.K.)

13.2.3 Vacuum pumps

The two types of rotary vacuum pump most commonly fitted to centrifugal wellpoint pump sets to assist priming are

1. The liquid ring or water recirculating vacuum pump
2. The flood-lubricated or oil-sealed vacuum pump

The vacuum units for the wellpoint application are required to operate continuously over a wide range of airflow rates from the very high to the very low. The service duty to be fulfilled is onerous. Generally, the vacuum pump is belt driven off the drive shaft of the prime mover. The energy consumption of the vacuum pump is of the order of 15% of the total of the motor output. The heat that may be generated can be considerable; thus, the cooling arrangements must be reliable. This requirement is particularly pertinent to the oil-sealed vacuum unit.

The liquid ring vacuum pump can better cope with low water-flow conditions, because the liquid ring pump recirculates some of the pumped groundwater, which is generally cool. Thus, the cooling requirements are less demanding than for the flood-lubricated vacuum pump. However, if it is expected that the pumped water will have a detrimental effect (e.g., if the groundwater is corrosive), special cooling modifications may be desirable, depending on the anticipated length of the pumping period and the chemical characteristics of the water to be pumped.

The liquid ring type is simpler than the flood-lubricated vacuum pump, but the degree of vacuum that it can generate is slightly less (about 5% lower). It is more robust and can be made capable of handling high airflow rates. The air handling capacity of the liquid ring pump is in the range of 1.4–14 m^3/min compared to a capacity of 1–3 m^3/min of the flood-lubricated pump, both at 0.85 bars and ambient temperature and pressure.

Adequate separation of air from the water in the float chamber is very important when a flood-lubricated pump is used. If the air is not adequately separated from the water, some water will be "carried over" to the vacuum pump and cause emulsification of the oil that forms the vacuum seal; this will cause damage to the vacuum pump.

13.2.4 Vacuum tank unit

The vacuum tank unit (Figure 13.3) is a variation of the float chamber, but of considerably greater volume (of the order of 5 m^3) and, therefore, is more efficient in separating the air from the water. A continuous vacuum can be applied to the tank by one or more vacuum pumps; this draws water to the vacuum tank, which gradually fills with water. The tank is fitted with one or more water pump(s) to discharge the water. Level sensors are used to

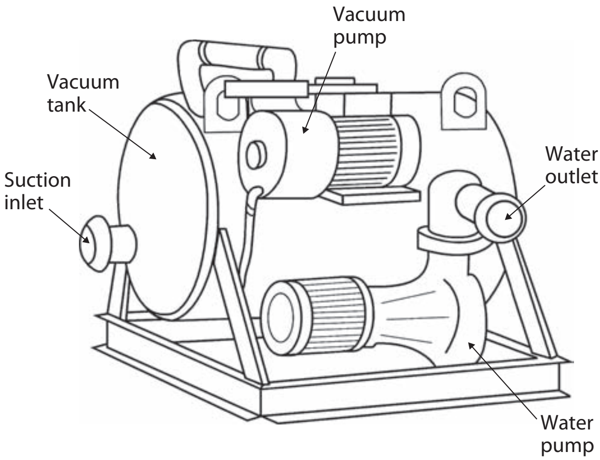


Figure 13.3 Vacuum tank unit for wellpointing.

switch the water pumps on or off, depending on the level of the water in the tank. Typically, the whole unit is electrically operated. The vacuum tank unit is efficient, especially when dealing with low rates of flow, because although the application of vacuum is continuous (and, in most units, the amount of applied vacuum can be varied, depending on the duty required), the energy for pumping the water is only mobilized as and when required to empty the tank. However, the sophistication of the total unit is considerable. These units are used frequently in continental Europe but are not widely used in the United Kingdom.

The vacuum tank unit is also useful for the ground improvement of very low-permeability soils, because by reducing the moisture content, the shear strength of the soil can be increased. The rate of increase in shear strength will be slow. The rate of pumping will tend to be very low, perhaps on the order of 100–200 L/h. A self-priming centrifugal pump could not cope with such a duty; it would grossly overheat within a relatively short period.

13.3 JETTING PUMPS

Wellpoint jetting units are high-pressure centrifugal clean-water pumps (often not self-priming) used for the installation of wellpoints and their risers, placing tubes, etc. Their outputs are usually in the range of 12–200 L/s at pressures of 4–23 bars. The most commonly used jetting pump for the installation of wellpoints is rated to deliver about 20 L/s at a pressure of about 6–8 bars.

13.4 UNITS FOR SUMP PUMPING

Sump pumping (see Chapter 8) is one of the most common pumping applications on construction sites. Many types of pumps are used for sump pumping; some are suitable, but some others are not. The sump pumping duty is most frequently needed to deal with surface water runoff. Unfortunately, usually little or no consideration is given to filtering of the water to be pumped. Thus, the units used for sump pumping must be able to cope with some suspended solids in the pumped water.

13.4.1 Contractor's submersible pump

A pump type commonly used for sump pumping is known as the contractor's submersible pump (Figure 13.4). It is an electric submersible unit with a sealed motor that usually runs in oil. The pump is of the bottom intake type (i.e., the water intake is beneath the motor). The pump is installed in a sump (Figure 8.2); the pumped water flows around the outside of the motor casing, thereby helping to keep the motor cool. Most units can operate on intermittent "snore," where the water level is drawn down to the pump invert, and the pump draws both air and water. It is prudent to suspend the unit so that the bottom of the pump intake is about 300 mm above the bottom of the sump; this allows for some accumulation of sediment in the sump. These units are less mechanically inefficient than some other types of pump. When high flows are involved, the energy costs are significant.

Figure 13.4 Contractor's electric submersible pump. (Courtesy of Geoquip Water Solutions, Ipswich, U.K.)

The submersible pump does not operate on the suction principle; thus, there is no limit to the possible drawdown, provided that pumps of sufficient power and head rating are used. Commonly used pumps range from 4.5 kW units with 75-mm outlets (capable of pumping around 10 L/s at a 10-m head) up to 40 kW units with 200-mm outlets (capacity of 100 L/s at a 25-m head). Hydraulic submersible pumps are also available, where the submersible pump is driven by a diesel hydraulic power pack located outside the sump at ground level.

13.4.2 All-purpose self-priming centrifugal pump

These units are very similar to pump sets used for wellpointing duties (see Section 13.2) but are adapted to be tolerant of some suspended solids in the pumped water. This requires the pump to be manufactured with generous clearance between the impeller and the volute casing to allow fines to pass. However, by allowing for extra clearance in the pump internals, some hydraulic efficiency is lost, and "all-purpose" pumps normally have a reduced performance compared with dedicated wellpoint units. As with units for wellpointing duties, pumps may be driven either by diesel or by electrical power.

13.5 PUMPS FOR DEEP WELLS

There are two types of pump commonly used to pump from deep wells:

1. The borehole electro-submersible turbine pump
2. The vertical lineshaft turbine pump

The pump end is similar in configuration in both types, but the drive is different. The submersible pump is driven by a submersible electric motor incorporated into a common casing with the pump (the wet end). The complete unit is positioned near the bottom of the well. The lineshaft pump unit (the wet end) is likewise positioned near the bottom of the well but is driven via a lineshaft by a motor (either electric or diesel) mounted at the surface.

13.5.1 Borehole electro-submersible turbine pumps

Generally, in developed countries, deep well pumps for temporary works projects are of the borehole electro-submersible turbine type. These are often used by the water supply organizations. Hence, a great variety of these are readily available, almost off-the-shelf in many instances. The submersible turbine pump has a high mechanical efficiency: 70% to 80% is common. Figure 13.5 is a photograph of a typical submersible turbine pump.

The borehole submersible units are slim and thus economize on the borehole and well screen diameter necessary to accommodate the pump.

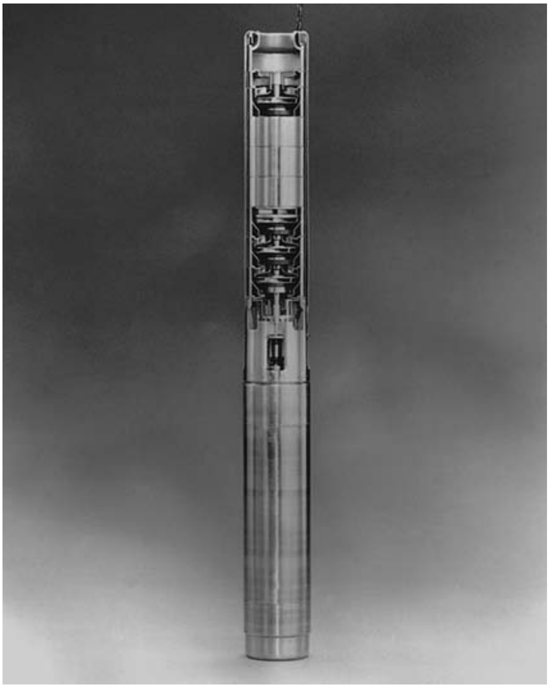


Figure 13.5 Borehole electro-submersible turbine pump. The pump shown is a cutaway demonstration version to show the shaft and impeller inside the wet end at the top part of the pump. (Courtesy of Grundfos A/S, Bjerringbro, Denmark.)

The smallest electro-submersible pumps in common use have a capacity of around 3 L/s and can be installed inside the well screen and casing of 110–125-mm-internal-diameter. Pumps of 10 L/s capacity can normally be installed inside a 152-mm-internal-diameter (the old imperial 6-in size) well screen and casing. Pumps of up to 40–80 L/s capacity are available and require a well liner and casing of 250–300 mm or larger internal diameter.

The pump manufacturer's specification will normally state the minimum internal well diameter into which a given model of pump can be installed and operated. This should be used as a guide, but remember that the manufacturer's recommendation will assume that the well is straight and true. In reality, the well casing may have been installed with slight twists or deviations from the plumb, and these may make the pump a tight fit when, in theory, it should pass freely. Pumps do sometimes become stuck and have to be abandoned down the well (resulting not only in the need for a new pump but also for a new well), much to the chagrin of all concerned. If cost and available drilling methods allow, it is good practice to slightly increase the diameter of the well liner and casing above the minimum required for the pump.

For each pump capacity, there is a range of pumps offering the same flow rate, but at increasing head. The additional head is achieved by adding additional stages of impellers to the wet end and the corresponding increase in the power requirements of the electric motor. Pumps are typically constructed largely from stainless steel, but some plastic, cast iron, or bronze components may also be used.

Most borehole submersible turbine pumps have their electric motor located in the lower part of the pump body casing with the wet end above. The wet end consists of the water intake and stator (these are both integral parts of the casing), and the impellers that are fixed to the drive shaft from the motor.

There is a waterproof seal on the pump shaft between the motor and the pump unit. Beneath the motor, there is a bottom bearing that is designed to take the weight and end thrust of the whole pump set. Most submersible turbine pump units supplied for deep wells have three-phase motors. However, single-phase units are available for low duties and outputs. They have motors of up to about 5 kW.

The pump is installed centrally in the well on the end of its riser pipe (Figure 13.6) after the well has been developed; thus, the pumped groundwater has no fines in suspension (see Section 10.7).

Figure 13.6 Lowering borehole electro-submersible pump and riser into the well. (Courtesy of Grundfos A/S, Bjerringbro, Denmark.)

In order to prevent water in the riser pipe from running back through the impellers into the well when the power is switched off, there should be a nonreturn valve immediately above the pump outlet. Such a backflow would be harmful if an attempt was made to restart the pump while the water in the riser pipe was still running back; under such circumstances, the additional start-up load could cause overloading. However, a nonreturn valve will mean that the riser pipe will remain full of water when the pump is switched off. This increases the weight to be lifted when the pump is removed from the well. It is acceptable to drill a 5-mm-diameter hole in the nonreturn valve to allow water to drain from the riser pipe sufficiently slowly.

13.5.2 Borehole electro-submersible pumps—Operational problems

13.5.2.1 Motor failure

There are three possible causes of motor failure:

1. Wear of the seal between the motor and the impellers. In time, wear would allow water to enter the sealed motor, thus causing failure of the motor windings.
2. Uneven wear of the bottom thrust bearing. This will lead to uneven wear of the upper seal, also causing failure of the motor windings.
3. Overheating of the motor. This will damage the windings of the motor.

13.5.2.2 Seal wear

In order to avoid failure due to seal wear, it is necessary to ensure that the pumped water is clean. However, if the grading of the filter media around the well screen is too coarse to retain the soil particles (or the well has not been adequately developed; see Section 10.7), the pumped water may contain fine particles in suspension. In time, this will cause wear of the impellers and stator, leading to reduced output. Moreover, the seal between the impellers and the motor will be worn, eventually causing failure of the motor windings. The rewinding of a pump motor is both costly and time consuming.

13.5.2.3 Bearing failure

It is important that the pump set hangs vertically. Otherwise, the thrust on the bottom bearing pad will be uneven, leading to uneven wear of the seal between the motor and the impellers; eventually, water will penetrate the windings of the electric motor, causing it to fail.

13.5.2.4 Motor overheating

In operation, the pump casing is immersed in the groundwater, and the motor windings are surrounded by a jacket filled with oil- or water-based emulsion coolant fluid. Provided that the flow to the well is significant compared with the volume of water surrounding the pump motor casing, overheating of the motor windings is unlikely. However, if the rate of pumping of the well is low (e.g., in a low-permeability soil), the pump motor casing may be in "dead water." The heat generated by the motor will gradually raise the temperature of the dead water in the well sump above the generally cool temperature of the inflowing groundwater. Eventually, the motor windings will overheat and burn out.

The risk of encountering dead water in the well sump is more prevalent, where the sump length of a well is formed on the top of an underlying clay layer. Oftentimes, it is necessary to form the sump section to penetrate into the clay to achieve maximum drawdown by positioning the pump intake beneath the level of the base of the aquifer. This is particularly relevant when the proposed formation level is close to an aquifer/clay interface and the wetted screen length per well is limited.

Oftentimes, the cause of motor overheating is simply that the capacity of the submersible pump is too large for the yield of the well. This is a problem that may occur in some hydrogeological conditions when well yields may decrease significantly after long periods of dewatering pumping. Ideally, in those circumstances, after the initial period of drawdown, the submersible pumps installed at the start of the project would be replaced with units of lower capacity. The labor cost of the pump change-out operation will be recovered in reduced energy usage over the years. This approach is described in more detail by Rea and Monaghan (2009).

Apart from changing to a smaller output pump, technically the most satisfactory course of action, there are two expedients that can be used when there is the risk of overheating of the motor windings. Fitting a shroud over the pump intake and motor casing will ensure that water from the cool groundwater source flows upward over the motor casing before reaching the intake and ensures that the motor is cooled (Figure 13.7a). An alternative is to fit a small bore (3 or 6 mm) bypass pipe tapped into the riser pipe above the pump outlet but below the nonreturn valve to divert some of the cool intake flow into the sump around and below the motor casing (see Figure 13.7b). This small and continuous flow of intake water will maintain cool water around the pump motor casing.

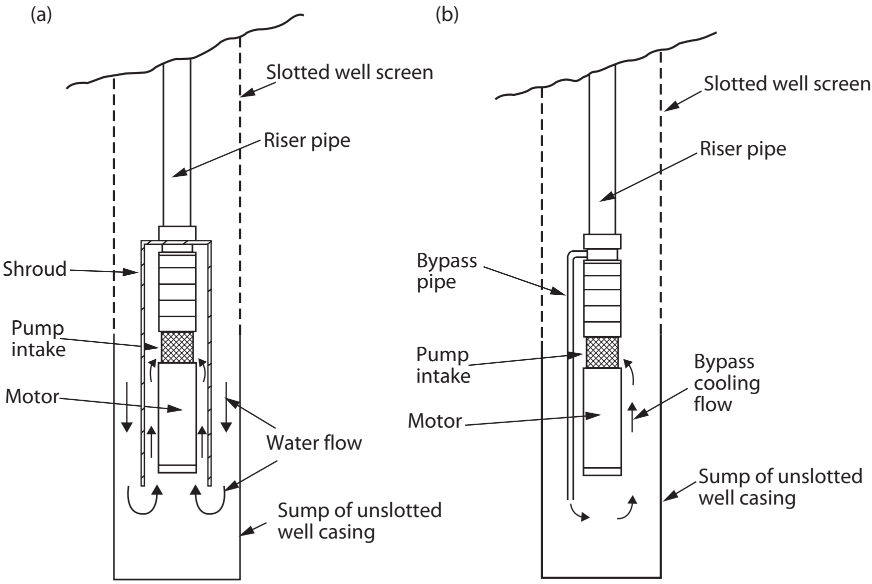


Figure 13.7 Methods of preventing overheating of borehole electro-submersible pump motors. (a) Motor shroud. The shroud is a tube, open only at its base, enclosing the electro-submersible pump. Water can only reach the pump intake by flowing over the motor, helping to keep it cool. (b) Bypass pipe. The small flow diverted from the riser pipe via a bypass pipe is discharged near the base of the motor to provide a cooling flow.

There is an insidious variation on motor overheating; although the discharge water may appear clear to the naked eye, this is no guarantee that there are no fines in suspension. When operating a low-yield well, there is a risk that the sump water is dead. Small fine particles, not visible to the naked eye, may settle in the dead water and gradually build up around the outside of the motor casing, with the result that there will be no sump water to cool the motor casing. In time, the windings will fail due to overheating. The fitment of either a shroud or a small bore bypass pipe, as described above, will prevent sediment buildup in the sump.

13.5.3 Vertical lineshaft turbine pumps

Lineshaft pumps (Figure 13.8) are particularly suitable for high-volume outputs at low heads and for high-horsepower duties. This type of pump

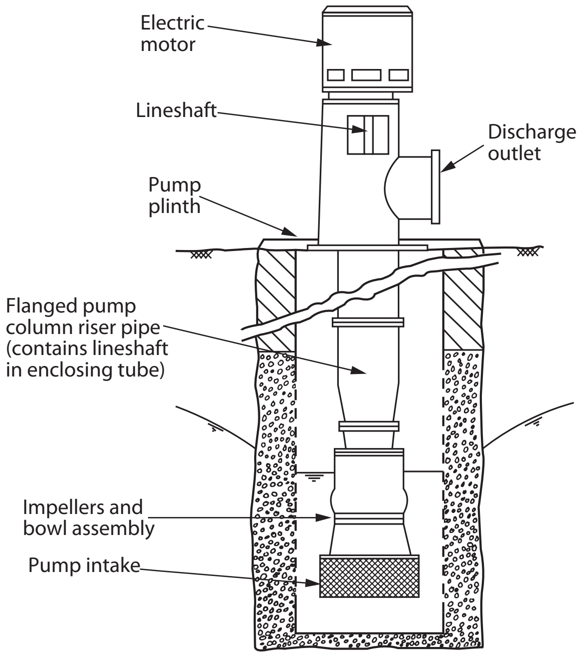


Figure 13.8 Vertical lineshaft turbine pump. The pump is powered by an electric motor mounted above the pump column.

is commonly used in developing countries, probably due to its greater simplicity in operation.

The wet-end unit of a lineshaft pump set (which consists of one or more impellers in a bowl assembly), similar to that of the submersible pump, is located near the bottom of the well. However, unlike the submersible pump set, the prime mover unit is located on top of or adjacent to the wellhead. The prime mover can be either petrol driven (small output pumps only) or diesel driven, with a separate energy source for each well. However, the prime movers may be electrically driven from a common power source.

The impellers of the pump unit are powered via a vertical lineshaft drive (with bearing assemblies) inside the pump column riser pipe, which is connected to the prime mover unit located at ground level. The verticality and straightness of the well is as important to the trouble-free operation of the lineshaft pump as for the submersible pump. The connection from the prime mover to the drive shaft depends on the type of prime mover and may be a direct coupling, a belt drive, or right angle gear. As with a borehole electro-submersible pump, it is prudent to incorporate a nonreturn valve at the pump outlet.

13.5.4 Lineshaft pumps—Operational problems

If the pumped water contains fines in suspension (either because the well was not developed adequately or the grading of the filter media is too coarse), then in time, the impeller stages and casing will wear and the pump output will decline. Otherwise, the lineshaft unit is generally trouble free and reliable in use, provided that the pump unit with its lineshaft assembly has been properly installed in the well, although there may be prime-mover troubles. All the foregoing assumes that the pump unit sizing is appropriate to the actual well yield.

13.5.5 Comparison of merits of lineshaft pumps versus electro-submersible pumps

The mechanical efficiency of lineshaft pumps tends to be slightly greater than that of submersible pumps. The initial cost of small output lineshaft pumps tends to be greater than a comparable-sized submersible pump, but this is reversed for the large output units.

The installation procedures for lineshaft pumps are more onerous. The well must be plumb. Moreover, skilled personnel are required for installation. Separate connections have to be made to each drive shaft length and pump column riser pipe joint assemblies, as these are being installed in the well. The standard length of shaft components may not be the same as those of the riser pipes; this can cause tedium in the installation of the riser pipe and drive shaft in the well. In contrast, the installation of an electro-submersible pump unit entails only the connections of the riser pipe; the electric power cable will be in one continuous length and is simply paid out as the pump is lowered into the well. However, a considerably lesser standard of skills is required for the maintenance and repair of lineshaft pumps as compared to the submersible unit. Motor or prime-mover repairs are particularly easy for lineshaft pumps because of the aboveground installation.

Each individual wellhead prime mover unit of a lineshaft pump installation should be visited at regular intervals to check on performance and fuel requirements. However, for a submersible pump installation, all the control switchgear, pump starters and associated process timers (for automatically restarting an individual pump if it trips out), and the changeover switchgear from the mains power to the standby generators (in case there is a failure in the mains supply, for whatever cause) can all be located inside a single switch house. This makes the supervision of running a deep well submersible pump installation easier and more straightforward, as well as less labor intensive.

13.6 SIZING OF PUMPS AND PIPEWORK

13.6.1 Sizing of pumps

On most groundwater lowering projects, standard or “off-the-shelf” pump sets will be used, either bought new, hired-in, or reused from a previous project. The pumps must be selected so that they can achieve the anticipated flow rate both once drawdown is established and during the initial period of pumping, when flow rates may be higher.

The flow rate produced by a given type of pump set varies (to a greater or lesser degree) with the “total head” against which the pump must act. In general, the greater the total head, the lower the output from a given pump set. Different types of pump will respond to changes in head in different ways; pump manufacturers produce performance charts that describe the idealized performance of each model of pump set.

Knowing the total head against which a pump set must work, the manufacturer’s performance charts allow the pump output under field conditions to be estimated. If the predicted pump output is less than that required, larger pump sets could be used, or (if the system design permits) multiple units could be employed.

The total head of pumping consists of three components: (1) suction lift; (2) discharge head; and (3) friction losses. The relationship between these components for submersible and suction pumps is shown in Figure 13.9.

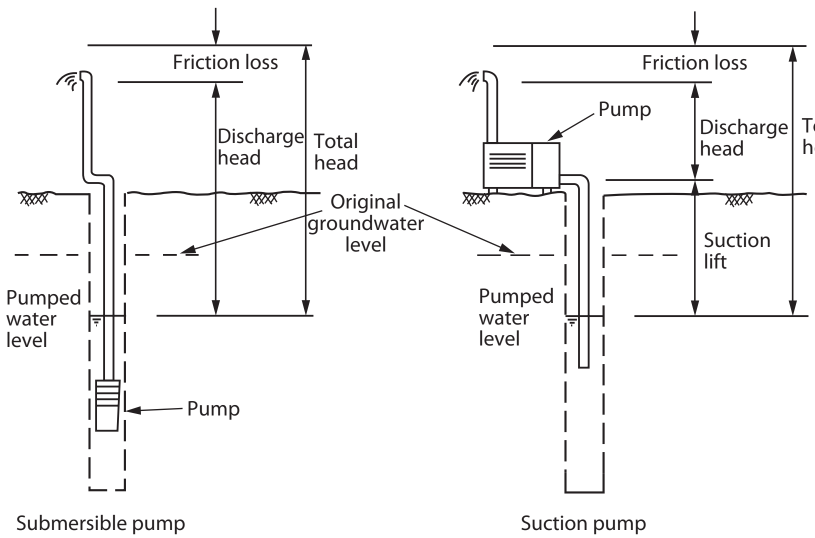


Figure 13.9 Discharge heads for pump sizing.

13.6.2 Discharge pipework

Discharge pipework is the system of pipes used to carry water from the pumps to the ultimate disposal point. It is a simple but vital part of any dewatering system and should not be neglected.

Due to the temporary nature of most dewatering systems, discharge systems most commonly comprise sectional pipework, delivered to the site in short lengths (typically less than 6 m but occasionally longer). Pipework is typically available in common diameters, often based on old imperial sizes. Common sizes for discharge pipework are nominal 6-in (150-mm) and 8-in (200-mm) diameter, although nominal 4- and 12-in diameter (100 and 300 mm, respectively) are also widely available. A wide range of bends, tees, valves, and reducers (to allow pipes of different sizes to be joined) are available and allow pipework to be laid to follow irregular routes or pass around obstacles (Figure 13.10).

Pipework can be made from a range of materials, although the most common materials used are steel and high-density polyethylene (HDPE). In most short-term applications, pipework is joined by flanged and bolted connections or proprietary "quick action" couplings fitted to the end of the pipes. On long-term or permanent projects, HDPE may be joined by butt welding or electrofusion welding.

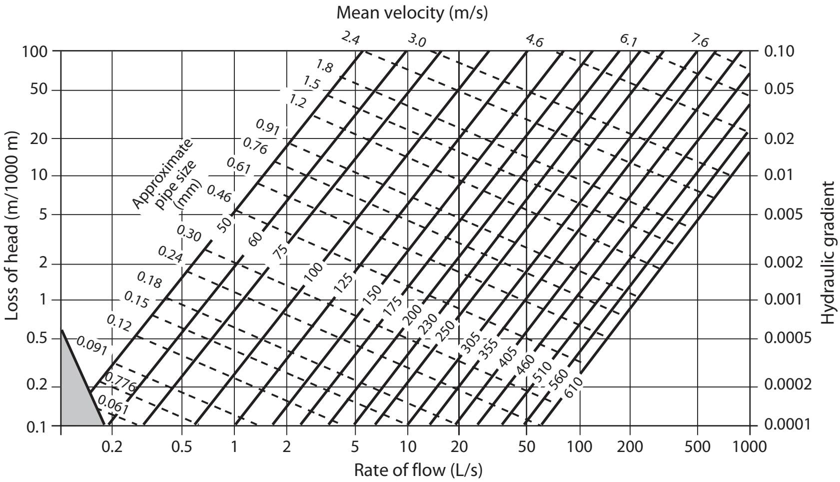
13.6.3 Friction losses

Friction losses are often small in comparison with the other components of the total head and are rarely a major issue in the design of dewatering systems using standard pump and pipework sizes. The main exception is in high flow rate systems (greater than around 50 L/s), where the water has to be discharged at considerable distances (greater than 100 m). In such cases, the friction losses may be significant and may reduce the output from the pumps. Figure 13.11 shows charts and tables (Preene et al. 2000) that allow friction losses to be estimated for commonly used pipe sizes and fittings.

If friction losses are perceived to be large enough to detrimentally affect pump outputs, then some remedial measures are possible.

1. Replace the pump sets with units rated at higher heads.
2. Provide additional pump sets so that a greater number of pumps producing a lower output can achieve the required total flow rate.
3. Modify the discharge pipework by either planning the pipework layout carefully to avoid unnecessary bends, junctions, or constrictions and using discharge pipework of larger diameter if available. If larger pipework is not available, one or more additional lines of discharge pipework could be laid, reducing the flow rate taken by an individual pipe.

Figure 13.10 Pipework used for dewatering discharges. (a) Sectional pipework used for dewatering discharge systems. (b) Pipe bridge used to carry dewatering discharge over access road. (Courtesy of Theo van Velzen BV, Heiloo, The Netherlands.)



Friction losses in valves and fittings as an equivalent length of straight pipe in meters

Type of fitting		Nominal pipe diameter (mm)								
		150	200	250	300	450	600	750	1065	1200
Gate valve	Open	1.1	1.4	1.7	2.0	2.8	4.3	5.2	6.4	7.6
	¼ closed	6.1	7.9	10.1	12.2	18.3	24.4	30.5	41.2	48.8
	½ closed	30.5	39.6	51.8	59.5	91.5	122.0	152.0	213.0	244.0
	¾ closed	122.0	159.0	213.0	244.0	366.0	488.0	610.0	854.0	976.0
Standard tee	Flow in line	2.9	4.3	5.0	5.9	9.1	11.9	15.1	22.0	24.7
	Flow to/from branch	9.8	12.8	16.8	19.8	30.5	39.6	50.3	73.2	82.3
Medium sweep 90° elbow		4.3	5.5	6.7	7.9	12.2	15.9	21.3	28.0	32.0
Long sweep 90° elbow		3.2	4.3	5.3	6.1	9.1	12.2	15.2	21.3	24.4
Square 90° elbow		9.8	12.8	16.8	19.8	30.5	39.6	50.3	73.2	82.3
45° Elbow		2.3	3.1	3.7	4.6	6.4	8.5	10.7	15.2	18.3
Sudden enlargement (→d→D→)	*d*/*D* = ¼	4.9	6.4	8.4	9.9	15.2	19.8	25.2	36.6	41.2
	d/*D* = ½	3.2	4.3	5.3	6.1	9.1	12.2	15.2	21.3	24.4
	d/*D* = ½	2.9	3.7	4.9	5.6	8.4	11.0	13.7	19.8	22.9
Sudden contraction (→D→d→)	*d*/*D* = ¼	2.3	3.1	3.7	4.6	6.4	8.5	10.7	15.2	18.3
	d/*D* = ½	1.7	2.3	2.9	3.4	4.9	6.4	8.2	11.3	12.8
	d/*D* = ½	1.1	1.4	1.7	2.0	2.8	4.3	5.2	6.4	7.6

Figure 13.11 Friction losses in pipework. (From Preene, M. et al., Groundwater control—Design and practice, CIRIA Report C515, Construction Industry Research and Information Association, London, 2000. Reproduced by kind permission of CIRIA: www.ciria.org.)

4. Reduce the distance that the discharge water must be pumped (e.g., by locating an alternative discharge point).
5. Provide "booster pumps" between the dewatering pumps and the discharge point. This will reduce the total head on the dewatering pumps, increasing their output.

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Chapter 14

Permanent groundwater control systems

14.1 INTRODUCTION

The vast majority of groundwater control applications are temporary. Dewatering or groundwater exclusion is carried out for a defined period, typically for the construction period of a structure or the production period of a mine or underground facility. The period for which groundwater control is required may be an extensive period, perhaps several months, even several years, but most systems are definitively temporary.

This chapter describes the issues associated with long-term groundwater control systems that are to be in operation so long such that they can be considered "permanent." There is no hard and fast definition of when a groundwater control system can be considered permanent, but for the purposes of this chapter, a system might be considered permanent if it is intended to operate for more than 5 to 10 years without major changes and is not part of an active construction or mining project. Permanent systems are typically an element of completed structures, facilities, or engineering infrastructure.

This chapter outlines the types of systems that can be used, describes the objectives of long-term or permanent groundwater control systems, and discusses practical problems.

14.2 TYPES OF PERMANENT GROUNDWATER CONTROL SYSTEMS

Similar to temporary systems, permanent groundwater control systems can be characterized as those based on the principles of groundwater pumping (Section 5.5) or exclusion (Section 5.4).

14.2.1 Permanent groundwater pumping systems

Systems based on pumping (Figure 14.1) work on the principle that groundwater is pumped (i.e., abstracted or removed from the ground) in order to

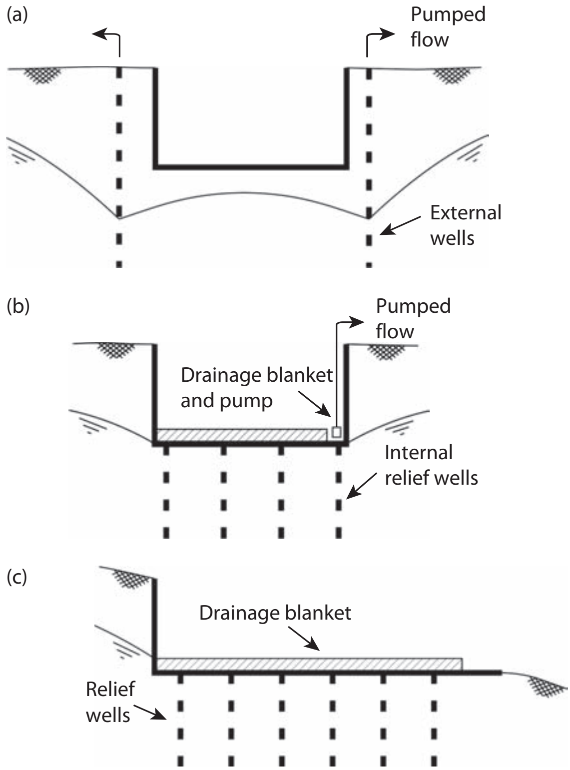

Figure 14.1 Permanent groundwater pumping systems. (a) Pumping system. The wells are pumped directly to lower groundwater levels. (b) Passive pumping system. The wells are not pumped directly, but are allowed to overflow to a drainage blanket, from where water is pumped away by a sump pump. (c) Fully passive pumping system. The topography of the ground is used to allow water to flow away from the wells or drainage features without the need for pumping.

lower groundwater levels or groundwater pressures. Systems may be based on conventional wells pumped by, for example, wellpoint or deep well systems (Figure 14.1a).

In some occasions, "passive" drainage systems may be used where the drainage elements such as relief wells (see Section 11.5) or drainage blankets are not pumped directly but have outlets at a sufficiently low level, relative to the original groundwater level, wherein water flows without the need for pumping. The water is then typically collected in sumps or drainage systems and pumped away (Figure 14.1b).

Where the terrain or geometry allows, such as in slope stabilization applications, it may be possible to allow the water to flow away by gravity, without the need for pumping (Figure 14.1c).

14.2.2 Permanent groundwater barrier systems

Barrier systems are used to exclude groundwater to reduce and, in some cases, to effectively eliminate seepage of groundwater into structures or other infrastructure that is below ground level. Examples of technologies used to form very low permeability groundwater cutoff barriers are described in Chapter 12, together with background on the advantages and disadvantages of this approach.

The most common application of barriers systems involves the use of artificial barriers (typically low-permeability vertical cutoff walls) in combination with natural “geological barriers” such as very low permeability layers of clay or unfractured rock (Figure 14.2a). In combination, the artificial and natural barriers act to form a complete exclusion system around the structure.

Where there is no suitable geological barrier, it is necessary to install an artificial barrier around the entire structure. This normally involves forming a very low permeability seal or “basal seal” beneath the structure, most commonly by some form of grouting method (see Section 12.9). Forming such a basal seal can be complex and expensive.

No barrier system is completely impermeable. Some residual seepage through the barrier should be anticipated, either through joints or imperfections in the barrier system or through the mass of the barrier. Some modest pumping capacity to drain any residual seepage is often associated with permanent barrier systems.

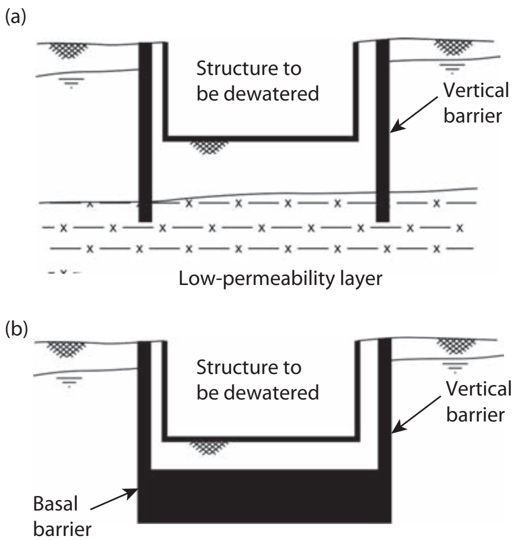

Figure 14.2 Permanent groundwater barrier systems. (a) Artificial barrier in combination with a geological barrier. (b) Artificial barrier with a basal plug.

14.3 OBJECTIVES OF PERMANENT GROUNDWATER CONTROL SYSTEMS

Permanent groundwater control systems can be used to meet a range of objectives, including

1. To reduce groundwater uplift loads on structures
2. To control anticipated rises in groundwater levels
3. To reduce leakage into belowground structures such as tunnels
4. To stabilize slopes and other earth structures
5. To form part of a system to remediate groundwater contamination

Each type of system is described in the following sections.

14.3.1 Reduction of groundwater uplift loads on structures

Any sealed structure constructed below groundwater level will be subject to groundwater pressures acting on its external faces. Where a structure is constructed to enclose a large void (such as an underground tank, water treatment plant, or Metro station) in permeable strata, the structure will experience a net uplift force and will be effectively buoyant (see Section 4.6).

If suitable engineering measures are not taken to counteract or reduce the net uplift force, there is a risk that the structure will become overstressed, may experience distortion or damage, or, in extreme cases, may physically rise up out of the ground until the buoyancy forces match the deadweight and other resisting forces. Any of these events would have serious consequences for the use of the structure.

One possible approach is to resist the uplift forces by the addition of deadweight to the structure (e.g., by increasing the thickness of concrete base slabs or walls) or by the use of structural elements such as tension piles or ground anchors to "hold down" the structure. An alternative approach is to use a permanent groundwater pumping system to lower groundwater levels and pressures so that there is no excess uplift pressure on the structure (Figure 14.3a).

Reduction in groundwater pressures is achieved by either pumped wells (located inside or outside the structure) or by relief wells flowing into a drainage blanket or gallery within the structure, from which water is pumped away.

Dry docks and certain other water- or fluid-filled structures present particular problems. In normal operations, these structures are full of water and are not buoyant, but at some point in their operational cycle, they will be emptied of fluid and will become buoyant. One solution is to have a

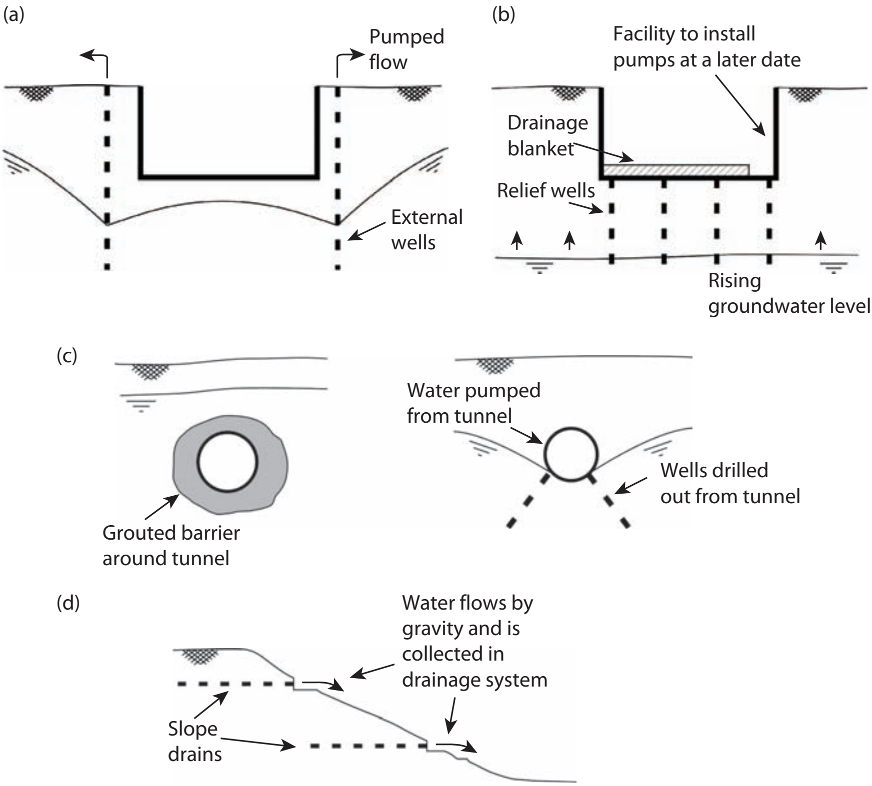

Figure 14.3 Objectives of permanent groundwater control systems. (a) Reduction of groundwater uplift loads on structures. Lowering of groundwater levels will result in a reduction of groundwater uplift forces on the structure. (b) Control of anticipated rises in groundwater levels. Wells and other drainage infrastructure are installed in anticipation of future rises in groundwater level. The system is not pumped following installation but has the capability to commence pumping at a later date if groundwater levels rise to problematic levels. (c) Control of leakage into belowground structures. Leakage can be reduced by forming a barrier around the structure or by pumping from within or around the structure. (d) Stabilization of slopes and earth structures. Slope drains are used to lower pore water pressures and to increase effective stress levels, therefore improving stability.

permanent dewatering system installed around the structure. The system is normally left "on standby" (i.e., fully or partly commissioned but not pumping), is pumped only when the structure is to be emptied, and is used to lower groundwater levels for the period until the structure is refilled. An alternative approach, sometimes used in dry docks, is to equip the base of the structure with an array of relief wells that will flow to pumped sumps when the structure is emptied. Care should be taken in the design of such systems to provide suitable seals or nonreturn devices on top of the relief wells to ensure that, when the structure is filled, the fluid inside does not leak out into the groundwater.

An example of the application of permanent dewatering system to address groundwater uplift loads is the Stratford Station Box in East London, as described by Whitaker (2004). The project involved the construction of a large underground structure to house a new station on the Channel Tunnel Rail Link (CTRL). A value engineering exercise identified such that a permanent dewatering solution would be a cheaper way of mitigating the effect of groundwater uplift pressures than structural alternatives such as tension piles. Subsequently, a system of 22 pumped wells was installed and commissioned to maintain groundwater levels at a predetermined level below the base of the station box structure.

14.3.2 Control of anticipated rises in groundwater levels

In many parts of the world, groundwater levels have been influenced by human activities and are subject to long-term change. The phenomenon of rising groundwater levels (see Section 3.6.1) is recognized in several major cities around the world (Wilkinson and Brassington 1991). This phenomenon typically occurs when water levels below cities have been historically lowered due to high levels of groundwater abstraction as the cities grow and develop. Rising groundwater occurs when abstraction quantities are reduced, either as a result of changing water use in the city or as a result of water quality and availability issues caused by the lowering of groundwater levels. The reduced levels of abstraction pumping allow groundwater levels to return slowly, probably over years or decades, to a higher level that is in equilibrium with the reduced abstraction regime. A classic example of this is the lower aquifer beneath London, as described by Simpson et al. (1989).

Rising groundwater levels can cause a range of problems for structures that are above groundwater level when first constructed but are below the levels to which groundwater levels are expected to recover to in the long term. Increased groundwater levels during the life of a structure can result in increased seepage into basements, tunnels, and subterranean structures; increased buoyancy forces; and changes in effective stresses (and, hence, ground strength) around the structure.

Where a structure is anticipated to be affected by rises in groundwater level, it may be necessary to include drains or relief wells in new structures (or retrofit them to existing facilities) to limit future groundwater rises to acceptable levels. Options include pumped wells located outside the structure or relief wells located inside the structure (Figure 14.3b). Nicholson and Harris (1993) give an example of relief wells installed in the basement of a new structure in London. Williams (2008) describes groundwater drainage measures installed in anticipation of rises in groundwater levels associated with impounding of water behind a tidal barrage constructed across Cardiff Bay, U.K. (Crompton and Heathcote 1993; Heathcote et al. 1997).

14.3.3 Reduction in leakage into belowground structures

It is not unusual for structures (such as basements or tunnels) that extend below groundwater level to "leak" and suffer from inflow of groundwater through imperfections in the structure. Structures may leak throughout their life (as a result of poor or inappropriate design and construction). More commonly, leakage begins to occur partway through a facility's life as a result of cracking or other deterioration in structural condition or in response to a change in external groundwater conditions (see Section 14.3.2).

It is relatively rare for groundwater leakage to completely inundate an existing structure; rates of groundwater leakage are often perceived to be merely a "nuisance" causing minor wet patches and puddles. However, there are many cases where apparently minor seepages have caused significant problems, for example, where sensitive goods are stored in basements or where electrical systems are used in railway tunnels.

Options for controlling the leakage into underground structures include exclusion (e.g., by using grouting to form low-permeability barriers around the structure) or pumping (by using wells inside or outside the structure). These methods are shown in Figure 14.3c.

There have been several cases where underground railway tunnels have suffered from problems of groundwater ingress, causing problems for electrical traction systems. Gallagher and Brassington (1993) describe groundwater problems on the Merseyrail tunnel system in the United Kingdom. A case history of a permanent dewatering system installed to control leakage into a tunnel on the Glasgow underground railway is given in Section 14.7.

14.3.4 Stabilization of slopes and earth structures

Reduction in pore water pressures by some form of drainage is widely used to improve the stability of slopes and earth structures, either as a planned design measure or as part of a remedial scheme following slope failure or ground movements.

The drainage effect reduces pore water pressures, thereby increasing effective stress and, in essence, improving soil strength. This means that interruptions in drainage flows can, in some cases, result in instability of the earth structure. Therefore, in many cases, passive drainage systems (where water flows by gravity) are used in preference to pumped systems, where interruption in pumping could cause instability issues (Figure 14.3d). Siphon drains (see Section 11.7) are one technology that can be used for slope stabilization, as described in the work of Bomont et al. (2005).

14.3.5 Remediation and containment of contaminated groundwater

Long-term control of groundwater, using pumping and/or exclusion technologies, can form part of schemes for remediating or containing contaminated groundwater (Figure 11.26). Further details of groundwater control technologies used on contaminated sites are given in Section 11.10. A case history of groundwater cutoff walls and pumping systems use as part of the remediation system at Derby Pride Park, U.K., is given in Section 11.10.4.

14.4 DESIGN ISSUES FOR PERMANENT GROUNDWATER CONTROL SYSTEMS

In principle, the design issues for permanent or long-term dewatering systems are no different from those for temporary systems with a relatively short-term life. Long-term systems still need to be designed to meet the project objectives and must be matched to the hydrogeological conditions at the site (see Chapter 7 for an overview of design issues).

However, the design process for permanent dewatering systems must address some additional issues that are not typically major concerns for short-term systems. Five of the most important issues are longevity, reliability, efficiency, environmental impacts, and legacy issues.

14.4.1 Longevity of permanent groundwater control systems

An obvious issue with permanent dewatering systems is that they must be designed and installed to a standard that will provide the system with a long life. Permanent systems must have operational lives measured in decades, which contrasts with most construction dewatering systems, where the operational life is measured in months.

With a little foresight, it is not difficult to design a dewatering system with a long life. This has long been the norm in the water supply industry, where public water supply wells and associated pumping systems are often designed for and routinely achieve operational lives in excess of 50 years. Longevity can be achieved by the application of basic engineering principles: (1) the selection of long-lasting, corrosion-resistant materials; (2) design to reduce the severity of chemical and bacterial clogging (see Section 16.9) and to allow access for cleaning and rehabilitation of wells and pipework;

(3) provision of redundancy in design where possible; and (4) design to allow easy replacement and upgrading of key elements of the system.

14.4.2 Reliability of permanent groundwater control systems

A dewatering system is a complex interconnected engineering system with inter alia mechanical, electrical, electronic, and human elements. If any of these elements fail to work correctly together, the dewatering system will effectively "fail." It is a significant challenge to keep a normal dewatering system operating reliably for a few months on a construction project. It is a much bigger challenge to ensure adequate reliability of a permanent dewatering system.

A key issue is to understand the system and identify any "bottlenecks" or key elements of the system. These are elements where the system would stop or would cease to be effective due to the damage or failure of a single element or component, where that element or component cannot be quickly or easily replaced or bypassed. It is sometimes not straightforward to identify the key bottlenecks. Power supply failure is an obvious key risk to the reliability of a dewatering system but is relatively easy to address by the provision of standby generators with automatic restart systems (see Section 16.7). In practice, some less obvious reliability issues can be harder to mitigate.

The reliability of water transmission and disposal is sometimes overlooked. For example, permanent dewatering systems often have their main water collection pipework buried so that it does not take up surface space and restrict access. This means that the pipework is out of sight; thus, there is a risk that, during the life of the system, perhaps the connection to the ultimate water disposal point (e.g., a municipal sewer) may become blocked or a pipeline might be accidently damaged, for example, by a contractor digging a new utility trench. The dewatering designer should consider this risk, and it may be appropriate to double or duplicate key sections of the pipework to allow rapid diversion of water flows in the event of damage or blockage in the pipework.

Measures for ensuring reliability can only be rationally developed if the implications and timing of "system failure" are fully understood. For example, how long can a system be inoperative before remedial action is needed? It makes more sense to invest in substantial backup systems in cases where groundwater levels recover rapidly (as might occur in a confined aquifer) compared to systems where water levels might take days or weeks to rise significantly following interruption in pumping. This assessment of failure issues should be carried out at the design stage and might be supplemented by data gathered during the commissioning and initial operation of the system. In some cases, a program of monitoring and testing is carried out during the commissioning period, including a "switch off test" wherein pumping is stopped in a controlled manner and the speed of recovery of water level is observed.

14.4.3 Efficiency of permanent groundwater control systems

For most temporary construction dewatering systems, the capital cost of equipment and the cost of installation will be much greater than the operating costs (e.g., power costs, spare parts, and other consumables). On most temporary dewatering projects, little effort is made to maximize the efficiency of operation, the focus often being on the speed of installation and the use of standardized equipment.

For permanent dewatering systems, the situation may be reversed. Total operating costs during the life of the system may have a much bigger impact on overall costs than capital costs. The designer may need to consider bespoke equipment and sophisticated control systems to get the desired performance at minimal operating costs.

One obvious way of reducing costs is to power pumps with electricity from the municipal grid system, because in urban areas, unit power costs tend to be cheaper than if generators are used for supply. Furthermore, it may be that the electricity tariff varies during a 24-h period, being cheaper at night when the demand is lowest. If pumps do not have to run continuously, it may be possible to reduce operating costs by running pumps preferentially at night and maximizing daytime idle periods when pumping is not needed.

Another strategy is to change the pumps for smaller units during the early part of the system life. If hydrogeological analysis indicates that the pumped flow rates will initially be high but will reduce after several weeks or months of pumping as aquifer storage depletion occurs (see Section 7.7.6), it may be appropriate to plan to use large pumps for initial pumping and then swap them for smaller pumps. The labor cost of the pump change-out operation will be recovered in reduced energy usage over the years. This approach is described in more detail by Rea and Monaghan (2009).

14.4.4 Environmental impacts of permanent groundwater control systems

Because permanent dewatering systems will be in operation for a long time, environmental impacts should be a potential concern. What may be an inconsequential impact from a temporary dewatering system in operation for 6 months may be a real issue of concern for a system operating for 20 years. The different types of possible impacts and methods of assessment are discussed in Chapter 15.

It is not unusual for numerical modeling to be used to predict the long-term impacts on the surrounding groundwater regime that may be caused by a permanent dewatering system. This is discussed further in Section 15.9.

14.4.5 Legacy issues for permanent groundwater control systems

For a typical temporary dewatering scheme for a construction project, throughout the operational life, there will always be someone involved in the project who is familiar with the dewatering system and knows some of its history and features. In contrast, there is a real risk that, in the later years of the life of a permanent dewatering system, it will become “forgotten about” and that the owners or occupiers of the site will have little detailed knowledge of the system—the staff will have left or retired, or the site may have been sold to new owners. Indeed, a reliable and smoothly operating permanent dewatering system may become a victim of its own success. If little maintenance or external intervention is required, detailed knowledge of the system will gradually be lost. This loss of knowledge can be a serious issue in the event of a future problem with the system when rapid intervention and repairs may be required. It can also cause problems if forgotten parts of the system are damaged or dug up, for example, during building works on the site.

The role of the dewatering designer is to try and ensure good communication with future site owners and occupiers (some of whom may not take possession of the site until decades hence). The communication can be via conventional means, such as an operation and maintenance manual containing simple and clear plans, diagrams, and instructions for the site owners. The dewatering designer can also communicate in a literally more concrete way by deliberately making pipework, wellheads, etc., visible where possible. Where equipment must be buried or covered over, substantial signs and markers can be set in walls and surfaces to show locations of important equipment.

14.5 PRACTICAL ISSUES FOR PERMANENT GROUNDWATER CONTROL SYSTEMS

In addition to the design issues described in the previous sections, permanent or long-term dewatering systems also face a range of practical challenges that must be overcome in order to ensure an effective and successful system.

14.5.1 Location of dewatering system

The location of wells and equipment is a fundamental issue for permanent dewatering systems. If the locations are not picked carefully, a range of problems can arise. The most obvious is that maintenance may be difficult, for example, if wells are located inside a structure where low headroom would restrict the use of cranes or lifting equipment to remove pumps. Another issue is simply that the elements of the dewatering system will take up valuable space that the site owner would rather use for business purposes. For that reason, dewatering wells and equipment are often placed in otherwise unproductive corners of the site or may be installed in subsurface chambers below areas such as car parks, which can be temporarily cordoned off when access is needed for maintenance.

14.5.2 Maintainability

A fundamental aspect of permanent dewatering systems is that they will be in operation for so long that every mechanical, electrical, or other active part of the system will require significant maintenance and, probably, replacement during the system's life. The dewatering designer should have designed an efficient and reliable system to minimize maintenance needs, but practical issues should also be addressed. For example

1. How will the owner or occupier know that maintenance or replacement of equipment is needed? Instead of having a responsive maintenance system where equipment is replaced when it fails, it may be better to have a monitoring system (see Chapter 16) that measures key parameters such as flow rates and water levels and will issue an alarm when key parameters reach critical values, possibly suggesting that the equipment is approaching the end of its life.
2. Can failed equipment be replaced quickly and easily? It is important to ensure that there is adequate access for lifting equipment and to physically transport large pieces of equipment (such as borehole pumps and rising main sections) to the work locations.
3. Can periodic well cleaning and rehabilitation (see Section 16.9) be carried out effectively? A key issue is can the dirty or discolored water typically generated by well rehabilitation be disposed of in an acceptable manner? The ultimate water disposal point should be assessed for its ability to accept dirty water. There may be a requirement to provide an alternative water disposal point during well rehabilitation or to provide room to use a mobile settlement tank (see Section 15.8.2) to reduce sediment loading in water pumped during rehabilitation.

14.5.3 Decommissioning

Nothing is truly permanent; thus, even very long-term dewatering systems will have a finite life or will ultimately no longer be needed if the structure or facility with which they are associated reaches the end of its useful life. It is important that some consideration is given to the requirements of decommissioning at the end of service life. Further details on decommissioning are given in Section 16.8.

14.6 OPPORTUNITIES ASSOCIATED WITH PERMANENT GROUNDWATER CONTROL SYSTEMS

Permanent dewatering systems that involve pumping are effectively a commitment to continued pumping of water and to the cost associated with it. Once the requirement for continued pumping is accepted, it can be appropriate to consider the potential opportunities—the potential to put the water to positive use. For example

1. The pumped water can be supplied (subject to water quality constraints) as raw water to be used for a range of purposes. The water may be suitable for use in irrigation systems or industrial processes. Subject to treatment by disinfection, the water may be passed into local and regional drinking water supply networks.
2. Brandl (2006) describes how wells used for groundwater lowering can also be an energy source by harvesting heat energy using the open-loop principle (Figure 3.2a), as described by Banks (2008), to provide heating and cooling for buildings and industrial processes.
3. In areas where rising groundwater is a potential concern, permanent dewatering systems can be incorporated into strategies to control regional groundwater levels. An example is the permanent dewatering system for the CTRL Stratford Station Box in London, which is intended to contribute to the strategy to control groundwater levels in the deep aquifer beneath London (Whitaker 2004).

14.7 CASE HISTORY: GOVAN UNDERGROUND TUNNEL, GLASGOW

The city of Glasgow, Scotland, has the third oldest underground railway in the world, dating from the 1890s. Now operated by Strathclyde Partnership for Transport (SPT), it forms an important part of the public transport network in the city; any prolonged interruption in its operation would be very disruptive to the citizens of Glasgow.

In the 1980s, part of the tunnel near Govan Station, where the layout was a twin tunnel with a crown and invert of concrete and brick sides, experienced inflows of water and fine sand through minor defects in the tunnel walls. The loss of material from beneath the tunnel caused settlement of around 75 mm, and that part of the railway had to be closed for several months while investigations and remedial works were carried out. The original remedial works included the installation of a permanent dewatering system, comprising vertical wellpoints through the trackbed (Figure 14.4a and 4b), pumped by wellpoint pumps located at track level (Figure 14.4c). The wellpoint system was effective in lowering piezometric levels in the silty fine sand materials around the tunnel, with pumped flow rates on the order of 20 L/s. One disadvantage of this layout is that, because the wellpoints and pump were located within the tunnels, maintenance could only be carried out during the short overnight track possessions when the railway was shut down.

The system of vertical wellpoints operated for more than 20 years but experienced some pump failures, resulting in soil loss into the tunnels. In later years, problems with the clogging of wellscreens and the buildup of iron-related deposits (see Section 16.9.2) in the wellpoints had reduced efficiency, and pumped water levels had risen, increasing the risk of further sand ingress into the tunnel. In the early 2000s, a scheme was developed to replace the existing wellpoints with a new system that would ensure the long-term stability of that section of the tunnel (Schünmann 2005).

A range of options were addressed at the design stage. Groundwater exclusion solutions based on grouting, either from surface or from inside the tunnel, were considered unsuitable due to the uncertainty of the condition of the tunnel and because of the disruption to railway operation associated with grouting from within the tunnel. Pumping systems based on new wells drilled out from inside the tunnel were deemed unsuitable for similar reasons.

The solutions investigated in most detail were based on lowering groundwater levels using a dewatering system located outside the tunnel. The sand deposits around the tunnel were relatively permeable, and the depth to tunnel invert was approximately 8 m below ground level. A conventional deep well system (see Chapter 10) would have been effective in lowering groundwater levels. However, although it was possible to find surface space and access for deep wells on a temporary basis, a system of permanent deep wells was not acceptable to the client due to the disruption to access to their surface operations. It was necessary to consider options with a much smaller surface footprint than conventional deep wells. Two options were proposed by the tendering contractors. One involved wells drilled by horizontal directional drilling (see Section 11.4) along the line of the tunnel, beneath the invert level. Another option was to use a small number of collector wells (see Section 11.6).

The solution that was finally adopted and installed on a design and construction basis by Keller Ground Engineering (with WJ Groundwater Limited as their subcontractor) included the installation of an array of four

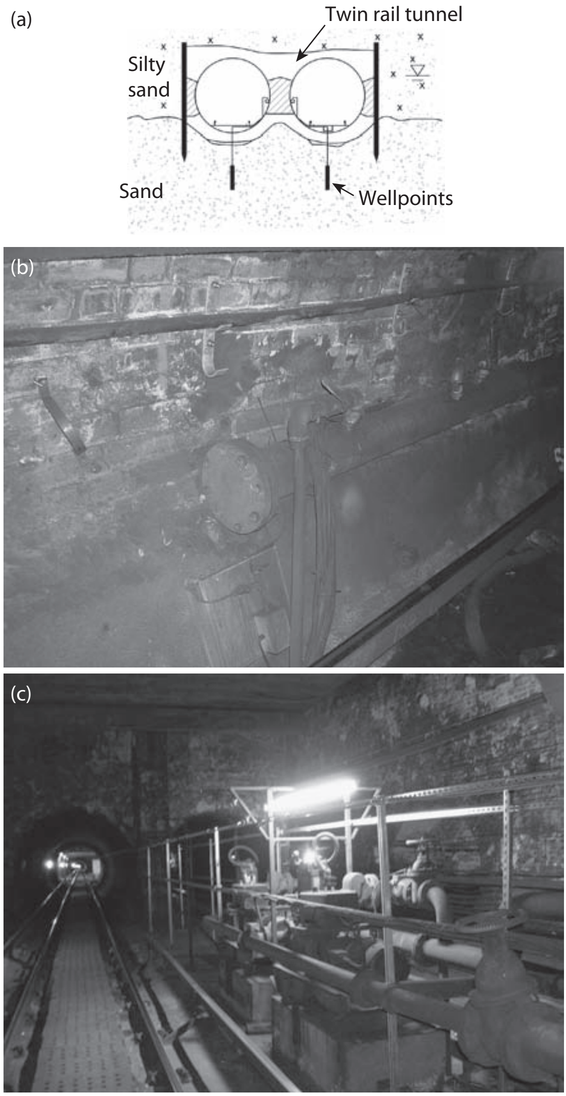

Figure 14.4 Internal wellpoint system within tunnel. (a) Cross section through an internal wellpoint system. A line of wellpoints is installed through the trackbed of each of the tunnels. The wellpoints are connected to header mains mounted within the tunnels. (b) Wellpoint header main mounted on internal face of railway running tunnel. The flexible connecting pipe to one of the wellpoints is visible in the foreground. (c) Wellpoint pumps within the tunnel. Electrically driven duty and standby wellpoint pumps are situated in an existing space at one end of the tunnels. The railway tracks and one of the tunnels are visible in the left of the picture. (Courtesy of Strathclyde Partnership for Transport, Glasgow, U.K.)

collector wells around the line of the tunnel. The collector wells were to be pumped to hold piezometric levels down along the length of the tunnel. Four segmentally lined shafts of 4-m internal diameter and up to 14 m deep were sunk to form the wells themselves, and a total of 32 lateral wells were drilled radially outward from the shafts to extend beneath the tunnel (Figure 14.5). When the water level in the shaft of the collector well is lowered by pumping, groundwater flows along the lateral wells and enters the main well shaft from where it can be removed.

An elegant part of the system was that the drilling rig for the lateral wells was also used to drill interconnecting pipes between the shafts. As a result, it was possible to pump the entire system using a single set of duty and standby pumps located in one shaft. Lowering the water level in the pumped shaft allows water from the other shafts to flow to the pumps by gravity along the interconnecting drains. The design flow rate for the system was 20–25 L/s, handled by a single duty electric submersible pump located in one shaft.

The drilling of the lateral wells presented practical challenges. Rotary drilling was carried out at a 127-mm diameter, with a maximum drilled length of 35 m (Figure 14.6). The original groundwater level was on the order of 10 m above the level of drilling out from the shaft. Considerable thought was given to the method of drilling the lateral wells out from the shafts. If drilling was carried out against the original external groundwater head, considerable volumes of water would enter the shaft and have to be pumped away, even if a stuffing box was used to try and seal around the rotating drill rods. This meant that there was a risk that fine sand could be washed into the tunnel, possibly causing problematic ground settlements. The solution adopted was to use a temporary dewatering system of conventional vertical deep wells to lower piezometric levels during drilling too close to the level of the lateral wells. The lateral wells were completed with a 60-mm-diameter slotted screen. To ensure that an effective filter could be installed, the screens had a prefitted resin-bonded sand filter of suitable grading (Figure 10.3c). This avoided problems with placement of filter media in horizontal boreholes. The temporary deep well system was decommissioned once the collector wells were in operation.

The construction contract included the responsibility for monitoring and maintenance of the system for the first 20 years of operation. Some of the old vertical wellpoints in the tunnel trackbed were converted to monitoring wells by the installation of vibrating wire pressure transducers linked to dataloggers (see Section 16.6) in the wells. Monitoring of tunnel condition and ground levels to check for settlement was also part of the monitoring program.

Unlike the previous wellpoint system, where any interruption in pumping resulted in almost immediate sand and water inflows to the tunnel, the new system holds groundwater levels sufficiently low, that if duty and

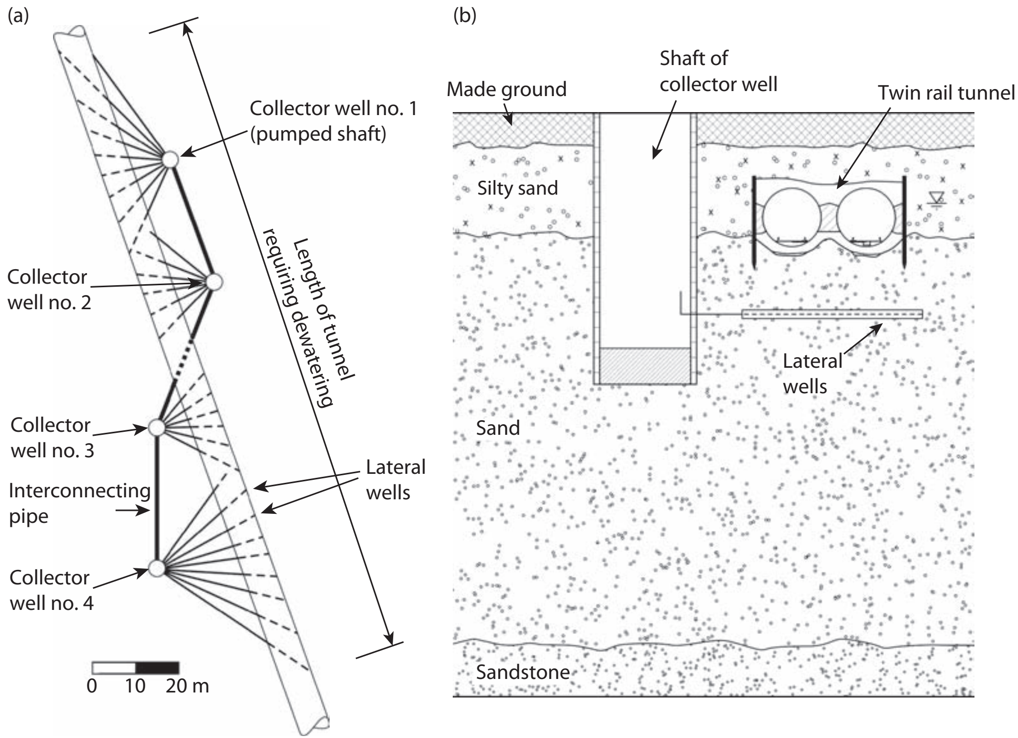

Figure 14.5 Collector wells installed around the tunnel. (a) Plan view. The tunnel length is dewatered by lateral wells drilled out from four collector well shafts. (b) Cross section. Lateral wells are drilled radially outward from the collector well shafts. (Courtesy of Strathclyde Partnership for Transport, Glasgow, U.K., and Keller Geotechnique, Wetherby, U.K.)

Figure 14.6 Installation of lateral wells by rotary drilling from within collector well shafts. Sump pumps are used to control the water from the drilling flush. (Courtesy of Strathclyde Partnership for Transport, Glasgow, U.K., and Keller Geotechnique, Wetherby, U.K.)

standby pumps fail, a period of 2 to 3 days elapses before water levels rise to the tunnel invert. This provides sufficient time to make arrangements for repair.

Based on the experience of well clogging from the old tunnel wellpoint system, the buildup of iron-related deposits in wells and pipework was anticipated and allowed for in the maintenance program. It was hoped that, because the lateral wells were kept permanently submerged under normal operating conditions, they would suffer less from iron-related clogging than the previous system, which from time to time had drawn in air. However, in the early years of operation, some clogging occurred, and an annual program of flushing of lateral wells has been instigated.

Chapter 15

Environmental impacts from groundwater control

15.1 INTRODUCTION

Groundwater control operations, whether using pumping or exclusion methods, have the potential for causing adverse impacts on the groundwater environment. These impacts may, in some circumstances, extend for some considerable distance beyond the construction site itself. In many cases, this will not cause problems, but there are situations when the potential impacts may be significant enough to merit careful monitoring and, if necessary, mitigation. This chapter outlines some potential environmental impacts that can result from groundwater control, the conditions in which they may occur, and possible mitigation measures.

The potential impacts discussed in this chapter are

1. Impacts that may result from the abstraction or pumping of groundwater. These include ground settlement, impacts on groundwater-dependent features such as rivers and wetlands, derogation or depletion of groundwater sources, and changes in groundwater quality, including movement of contamination plumes and saline intrusion.
2. Impacts that may result from the creation of artificial groundwater pathways, such as poorly sealed boreholes.
3. Impacts that may result when low-permeability groundwater barriers (such as cutoff walls) are created.
4. The impact of discharges (such as artificial recharge or pollution leaks) on the groundwater environment.
5. The impact of discharge flows on the surface water environment.

15.2 WHY ARE IMPACTS FROM GROUNDWATER CONTROL OF CONCERN?

This book is primarily aimed at engineers and construction professionals who need to gain an understanding of how groundwater might affect belowground engineering works and how it can be suitably controlled by means of pumping and exclusion. In this context, most engineers would view groundwater as a *problem*. The presence of groundwater in water-bearing strata will bring complications and additional cost compared to a comparable but "dry" site. In effect, most people working in construction would, consciously or unconsciously, view groundwater as a "bad thing."

In stark contrast, professionals (such as hydrogeologists) working in water resources or environmental protection view groundwater quite differently. To them, groundwater is a "good thing." It is a *resource*—worth protecting and managing. In many countries around the world, groundwater is an essential source of water for direct human consumption, agricultural use, industrial processes, and a myriad of other purposes. Taking the United Kingdom as an example, in England, around one-third of public water supply is obtained from groundwater. However, these figures can hide considerable local variations. Even in regions with low overall groundwater usage, there may be communities that are dependent on groundwater for most or all of their water supply. In addition to groundwater abstractions for public supply, groundwater is relied upon by many rural families and communities for domestic water supplies from private springs or boreholes.

In addition to its value as a resource for water supply, groundwater has a strong interaction with many surface water features such as rivers and wetlands. Consequently, changes in groundwater levels or quality can have detrimental environmental impacts. In the United Kingdom, like in many other countries, groundwater protection policies, such as *Groundwater Protection: Policy and Practice* (Environment Agency 2008), have been applied by governmental regulatory bodies to prevent

1. Overabstraction of aquifers.
2. Derogation of individual sources.
3. Damage to environmental features dependent on groundwater levels (e.g., river baseflows).
4. Unacceptable risk of pollution of groundwater from point and diffuse sources. This includes delineation of source protection zones (SPZs) around individual groundwater abstraction sources, within which various activities (including some techniques used for groundwater control) are either strictly controlled or prohibited.

15.3 POTENTIAL ENVIRONMENTAL IMPACTS FROM GROUNDWATER CONTROL

Activities that are either directly or indirectly associated with groundwater control can cause a range of environmental impacts. These impacts can be grouped into five main categories (according to Preene and Brassington 2001):

1. Pumping or abstraction from aquifers
2. Physical disturbance of aquifers creating pathways for groundwater flow
3. Physical disturbance of aquifers creating barriers to groundwater flow
4. Discharges to groundwaters
5. Discharges to surface waters

The categories of impacts are summarized in Table 15.1 and described in more detail in the following sections.

Table 15.1 Potential environmental impacts from groundwater control activities

	Category	*Potential impacts*	*Duration*	*Relevant activities*
1	Abstraction	Ground settlement Derogation of individual sources Effect on aquifer—groundwater levels Effect on aquifer—groundwater quality Depletion of groundwater-dependent features	Temporary	Dewatering of excavations and tunnels using wells, wellpoints, and sumps Drainage of shallow excavations or waterlogged land by gravity flow
			Permanent	Permanent drainage of basements, tunnels, and road and rail cuttings, both from pumping and from gravity flow
2	Pathways for groundwater flow	Risk of pollution from near-surface activities Change in groundwater levels and quality	Temporary	Vertical pathways created by site investigation and dewatering boreholes, open excavations, and trench drains Horizontal pathways created by trenches, tunnels, and excavations
			Permanent	Vertical pathways created by inadequate backfilling and sealing of site investigation and dewatering boreholes and excavations and by permanent foundations, piles, and ground improvement processes Horizontal pathways created by trenches, tunnels, and excavations

(*continued*)

Table 15.1 (Continued) Potential environmental impacts from groundwater control activities

	Category	*Potential impacts*	*Duration*	*Relevant activities*
3	Barriers to groundwater flow	Change in groundwater levels and quality	Temporary	Barriers created by temporary or removable physical cutoff walls such as sheet piles or artificial ground freezing
			Permanent	Barriers created by reduction in aquifer hydraulic conductivity (e.g., by grouting or compaction)
4	Discharge to groundwaters	Discharge of polluting substances from construction activities	Temporary	Leakage and runoff from construction activities (e.g., fuelling of plant) Artificial recharge (if used as part of the dewatering works)
			Permanent	Leakage and runoff from permanent structures Discharge via drainage soakaways
5	Discharge to surface waters	Effect on surface waters due to discharge water chemistry, temperature, or sediment load	Temporary	Discharge from dewatering systems
			Permanent	Discharge from permanent drainage systems

Source: After Preene, M., and Brassington, F.C., The interrelationship between civil engineering works and groundwater protection, Protecting Groundwater, Project NC/00/10, Environment Agency, National Groundwater and Contaminated Land Centre, Solihull, 2001, pp. 313–320.

15.4 IMPACTS FROM GROUNDWATER ABSTRACTION

Abstraction or pumping of groundwater is a natural part of most groundwater lowering systems. Groundwater is typically pumped on a temporary basis. Occasionally, dewatering systems operate for long enough that they can be considered "permanent" (see Chapter 14).

A number of groundwater impacts may result from groundwater abstraction for dewatering purposes. These include

1. Ground settlement
2. Depletion of groundwater-dependent features
3. Effects on water levels and water quality in the aquifer as a whole
4. Derogation of individual borehole or spring sources
5. Other, less common, effects

15.4.1 Settlement due to groundwater lowering

Ground settlements are an inevitable consequence of every groundwater lowering exercise. In the great majority of cases, the settlements are so small such that no distortion or damage is apparent in nearby buildings. However, occasionally, settlements may be large enough to cause damaging distortion or distress of structures, which can range from minor cracking of architectural finishes to major structural damage. In extreme cases, these effects have extended several hundred meters from the construction site itself and have affected large numbers of structures.

If there is any concern that groundwater lowering (or any other construction operation) may result in ground settlements beneath existing structures, it is essential that a preconstruction building condition survey be carried out. This exercise (sometimes known as a dilapidation survey) involves recording the current state of any structures that may be affected by settlement. This should provide a detailed record of any preexisting defects so that, if damage is alleged, there is a basis for judging the veracity of the claims. Unfortunately, for groundwater lowering projects, where the influence of the works may extend for several hundred meters from the site, the building condition survey area selected is often too small and does not cover all the structures that may be significantly affected. Ideally, the extent of the survey should be finalized following a risk assessment exercise.

Settlements caused by groundwater lowering may be generated by a number of different mechanisms, some easily avoidable, some less so:

1. Settlement resulting from the instability of excavations when groundwater is not adequately controlled
2. Settlement caused by loss of fines
3. Settlement induced by increases in effective stress

15.4.2 Settlement due to poorly controlled groundwater

This book describes the philosophy and methods whereby groundwater can be controlled to provide stable excavations for construction works. However, groundwater is sometimes not adequately controlled, leading to the instability of excavation, uncontrolled seepages, and perhaps base failure (sometimes called a groundwater "blow"; see Section 4.6). These problems may result from several causes, such as failure to appreciate the need for groundwater control, a misdirected desire to reduce costs by scaling down or deleting groundwater control from the temporary works, inadequate standby or backup facilities to prevent interruption in pumping, and ground or groundwater conditions not anticipated by the site investigation or design and not identified by construction monitoring.

Ideally, with adequate investigation and planning, most of these issues can normally be avoided, especially the last one. Sadly, these problems still occur, leading to the failure of excavations and both significant additional costs and delays to the construction project (see the work of Bauer et al. 1980 and Greenwood 1984 for case histories). If there is a sudden "blow" or failure of an excavation, soil material will be washed into the excavation. This can create large and unpredictable settlements around the excavation, much larger than the effective stress settlements associated with groundwater control. Any buildings in the area where the uncontrolled settlements occur are likely to be severely damaged.

15.4.3 Settlement due to the loss of fines

Settlement can also occur if a groundwater lowering system continually pumps "fines" (silt- and sand-sized particles) in the discharge water—a problem known as "loss of fines." Most dewatering systems will pump fines in the initial stages of pumping, as a more permeable zone is developed around the well or sump (see Well Development, Section 10.7). However, if the pumping of fines continues for extended periods, the removal of particles will loosen the soil and may create subsurface erosion channels (sometimes known as "pipes"; see Section 4.6.4). Compaction of the loosened soil or collapse of such erosion channels may lead to ground movements and settlement.

Continuous pumping of fines is not normally a problem with wellpoints, deep wells, or ejectors, provided that adequate filter packs have been installed and monitored for fines in their discharge. Occasionally, a sand pumping well may be encountered, perhaps caused by a cracked screen or poor installation techniques. Such wells should be taken out of service immediately.

The method that most commonly causes the loss of fines is sump pumping (see Chapter 8). This is because the installation of adequate filters around sumps is often neglected, allowing fine particles in the soil to become mobile as groundwater is drawn toward the pump. Section 8.7 shows an example of the problems this can cause. Powers (1985) lists soil types where the use of sump pumping is fraught with risk. These include

1. Uniform fine sands
2. Soft, noncohesive silts and soft clays
3. Soft rocks where fissures can erode and enlarge due to high water velocities
4. Rocks where fissures are filled with silt, sand, or soft clay, which may be eroded
5. Sandstone with uncemented layers that may be washed out

In these soil types, even the best engineered sump pumping systems may encounter problems. Serious consideration should be given to carrying out groundwater lowering by a method using wells (wellpoints, deep wells, or ejectors) with correctly designed and installed filters.

15.4.4 Settlement due to increases in effective stress

Lowering of groundwater levels will naturally reduce pore water pressures and, hence, increase effective stress (see Section 4.3). This will cause the soil layer to compress, leading to ground settlements. In practice, however, for the great majority of cases, the effective stress settlements are so small such that no damage to nearby structures results.

The magnitude of effective stress settlements will depend on a number of factors:

1. The presence and thickness of a highly compressible layer of soil below the groundwater level, which will be affected by the pore water pressure reduction. Examples include soft alluvial silts and clays or peat deposits. The softer a soil layer is (and the thicker it is), the greater the potential settlement becomes.
2. The amount of drawdown. The greater the drawdown of the groundwater level, the greater the resulting settlement.
3. The period of pumping. In general, at a given site, the longer the pumping is continued, the greater the settlement becomes.

Powers et al. (2007) state that the most significant of these factors is the presence of a layer of highly compressible soil. It is certainly a truism that damaging settlements are unlikely to result from groundwater lowering on sites where highly compressible soils are absent. The corollary of this is that the potential for damaging settlements should be investigated carefully on any sites where there is a significant thickness of highly compressible soils.

Effective stress settlements can be calculated using the basic soil mechanics theory. The final (or ultimate) compression ρ_{ult} of a soil layer of thickness D is

$$\rho_{ult} = \frac{\Delta u D}{E'_o} \tag{15.1}$$

where Δu is the reduction in pore water pressure, and E'_o is the stiffness of the soil in one-dimensional compression (which is equal to $1/m_v$, where m_v is the coefficient of volume compressibility). E'_o can be estimated using several techniques; values used in calculations should be selected with care.

Equation 15.1 is based on the assumption that total stress remains constant as the groundwater level is lowered. This is a reasonable simplifying assumption, because the difference between the unit weight of most soils in saturated and unsaturated conditions is generally small and, given the uncertainties in other parameters, can be neglected without significant error.

To be useful in practice, Equation 15.1 needs to be written in terms of drawdown s. This will be different for aquicludes, aquifers, and aquitards (see Section 3.4).

1. *Aquicludes*. This type of stratum is of very low permeability. During the period of pumping from an adjacent aquifer, no significant pore water pressure reduction (and, hence, compression) will be generated.
2. *Aquifers*. This type of stratum is normally of significant permeability and is pumped directly by the wells or wellpoints. Pore water pressure reductions will occur effectively at the same time as the drawdown. In practice, this means that the compression and settlement of aquifers occurs instantaneously once drawdown occurs. The rate at which drawdown occurs and the resulting pattern of drawdown around a groundwater lowering system can be estimated using the methods given in Section 7.9.
3. *Aquitards*. These strata are at least one to two orders of magnitude less permeable than aquifers and are not generally pumped directly by wells or wellpoints. Aquitards will tend to drain vertically into the aquifer, at a rate controlled by the vertical permeability of the aquitard. In practice, this means that, even if the drawdown and compression of the aquifer has stabilized, water may still be draining slowly out of the aquitard and will continue to do so until the pore water pressures equilibrate with those in the aquifer. The pore water pressure reductions and compressions of aquitard layers will tend to lag behind the aquifer and may take weeks, months, or even years to reach their ultimate value. Of course, if the groundwater lowering system operates for only a short period of time, the ultimate compression will not develop fully.

Figure 15.1 shows that, where aquifers and aquitards are present, the distribution of pore water pressure reduction with depth will depend on the nature of the strata at the site and will change with time. This should be taken into account when estimating ground settlements.

Settlements caused by groundwater lowering will generally increase with time, and will be greatest at the end of the period of pumping. There are two separate time-dependent effects at work.

1. *Increasing aquifer drawdown with time*. When groundwater is pumped from an aquifer by a series of wells, a zone of drawdown propagates away from the system at a rate controlled by the pumping

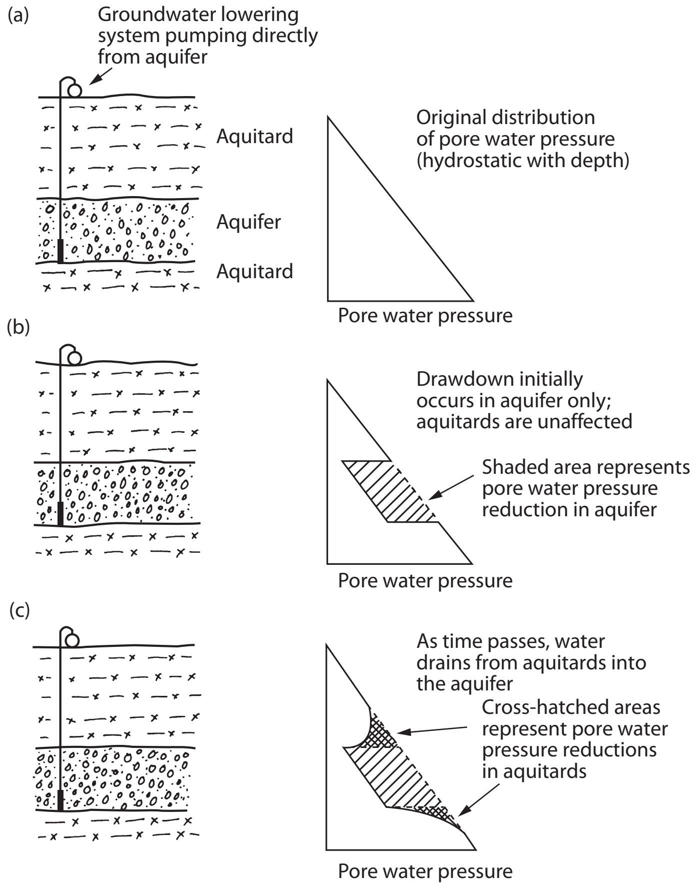

Figure 15.1 Pore water pressure reductions in response to groundwater lowering. (a) Prior to pumping. Hydrostatic conditions prevail in the aquifer and aquitards. (b) After short-term pumping. Pore water pressure reductions have not yet occurred in the aquitards. (c) After long-term pumping. Drainage from aquitards is occurring; as pumping continues, the pore water pressure reduction will propagate further from the aquifer into the aquitards.

rate and aquifer properties. This means that, at a given point (some distance from the system), drawdown in the aquifer will increase with time, as will the compression of the aquifer. For all practical purposes, the compression of an aquifer occurs contemporaneously with drawdown.

2. *Slow drainage from aquitards*. When drawdown occurs in an aquifer, any adjacent aquitards will begin to drain vertically into the aquifer. This drainage may occur quite slowly. It follows that the compression of aquitards will lag behind the drawdown in the aquifer.

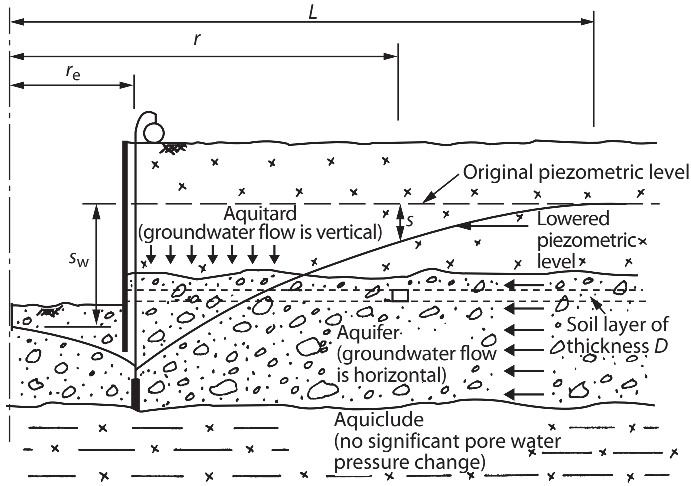

Figure 15.2 Groundwater flow to fully penetrating wells.

For a simple case (Figure 15.2) with wells fully penetrating a confined or semiconfined aquifer, groundwater flow will be horizontal, Δu will be constant with depth, and $\Delta u = \gamma_w s$, where γ_w is the unit weight of water, and s is the drawdown of the piezometric layer, giving the ultimate compression of a soil layer as

$$\rho_{ult} = \frac{\gamma_w s D}{E_o'} \tag{15.2}$$

At the location under consideration, drawdown s can be estimated *approximately* using standard solutions for the shape of drawdown curves, such as those given in Section 7.9.

Equation 15.2 shows the ultimate compression. However, because compression may lag behind aquifer drawdown, settlement calculations should concentrate on the effective compression ρ_t at a time t after pumping commenced. For an aquifer, compression occurs rapidly; thus, the effective compression is the ultimate compression

$$(\rho_t)_{aquifer} = \rho_{ult} \tag{15.3}$$

However, the effective compression of an aquitard layer may be less than the ultimate compression

$$(\rho_t)_{aquitard} = R\rho_{ult} \tag{15.4}$$

where R is the average degree of consolidation (with a value of between zero and one) of the aquitard layer, determined in terms of a nondimensional time factor T_v, as shown in Figure 15.3.

The ultimate compression calculated in Equation 15.2 is based on pore water pressure reductions consistent with a confined aquifer, where the lowered piezometric head is not drawn down below the top of the aquifer. However, for greater drawdowns in confined aquifers, for unconfined aquifers, and for aquitards, the pore water pressure distribution will be different. Therefore, the effective compression estimated from Equations 15.2–15.4 needs to be converted to corrected compression ρ_{corr}

$$\rho_{corr} = C_d \rho_t \tag{15.5}$$

where C_d is a correction factor for effective stress (with values between 0.5 and 1.0), as shown in Figure 15.4. Note that this correction will tend to reduce the compression compared to the uncorrected values.

The above calculations are for the compression of individual soil layers. The result of interest to the designer is the resulting ground settlement ρ_{total}. This is determined by summing the corrected compressions of all aquifer and aquitard layers affected by the groundwater lowering

$$\rho_{total} = (\rho_{corr})_{\text{all aquifers}} + (\rho_{corr})_{\text{all aquitards}} \tag{15.6}$$

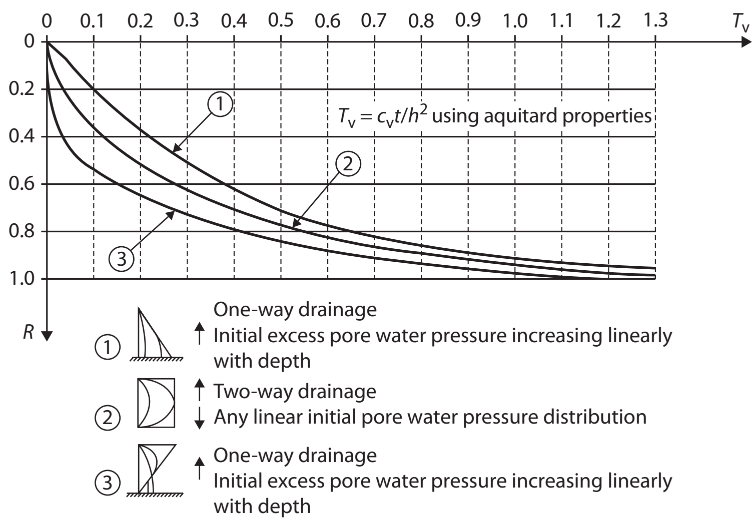

Figure 15.3 Average degree of consolidation of a soil layer vs. time. Note: $T_v = c_v t/h^2$ using aquitard parameters, where c_v = coefficient of consolidation, t = time since pumping began, and h = length of drainage path. (After Powrie, W., *Soil Mechanics: Concepts and Applications*, Spon Press, London, 2004.)

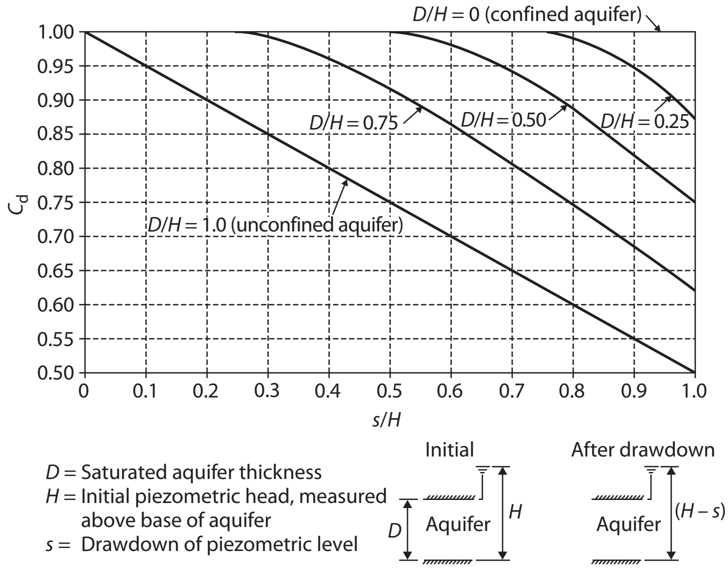

Figure 15.4 Correction factor for effective stress. (From Preene, M., *Proceedings of the Institution of Civil Engineers—Geotechnical Engineering*, 143(4), 177–190, 2000. With permission.)

15.4.5 Settlement damage to structures

From a practical point of view, it is necessary to consider the magnitude of settlement that will result in varying degrees of damage; predicted settlements will be of less concern if they will not damage nearby structures. Considerable work has considered the damage to structures due to self-weight settlements (Burland and Wroth 1975) or due to tunneling-induced settlements (Lake et al. 1996). There is a paucity of data on damage that may result from groundwater lowering-induced settlements. This may be because preconstruction building condition surveys are often not carried out over a wide-enough area around a groundwater lowering system; damaging settlements may occur up to several hundred meters away. If damage occurs (or is alleged by property owners), the actual damage caused by groundwater lowering can be difficult to assess, because the original condition of the property is not known. This is in contrast to settlements from tunneling or deep excavation, which rarely extend more than a few times the excavation depth; building condition surveys normally cover almost all of the structures at risk.

Structures are not generally damaged by settlements per se, rather by differential settlement or distortion across the structure. In uniform soil conditions, the typical convex upward drawdown curve will create a settlement profile that will distort structures in hogging (Figure 15.5). The drawdown

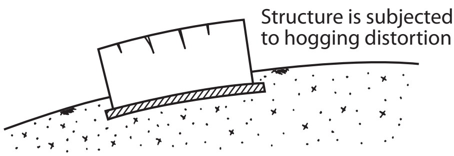

Figure 15.5 Deformation of structure due to settlement profile in uniform conditions. (From Preene, M., *Proceedings of the Institution of Civil Engineers—Geotechnical Engineering*, 143(4), 177–190, 2000. With permission.)

curve will propagate away from the groundwater lowering system with time; at a given location, the slope of the drawdown curve (and, hence, the differential settlements and distortion) will increase while pumping continues.

In fact, most groundwater lowering operations in uniform soils do not cause damaging settlements (unless the soils are very compressible). This is because the distortions and ground slopes resulting purely from the drawdown curve are generally slight. Variations in soil conditions or foundation type can allow more severe differential settlements or distortions to occur (Figure 15.6).

Damage risk assessment exercises are often carried out for tunnel projects by assessing the maximum settlement and tilt that a structure will experience (Lake et al. 1996). A similar approach can be applied to groundwater

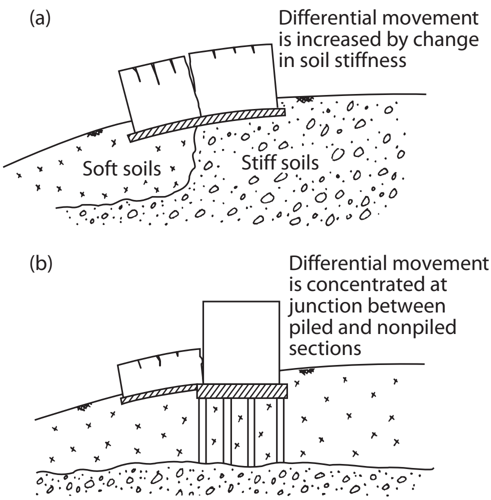

Figure 15.6 Deformation of structure due to settlement profile in nonuniform conditions. (a) Change in soil conditions. (b) Change in foundation type. (From Preene, M., *Proceedings of the Institution of Civil Engineers—Geotechnical Engineering*, 143(4), 177–190, 2000. With permission.)

Table 15.2 Tentative limits of building settlement and tilt for damage risk assessment

Risk category [a]	*Maximum settlement (mm)* [b]	*Building tilt* [c]	*Anticipated effects*
Negligible	<10	<1/500	Superficial damage unlikely
Slight	10–50	1/500–1/200	Possible superficial damage; unlikely to have structural significance
Moderate	50–75	1/200–1/50	Expected superficial damage and possible structural damage to buildings; possible damage to rigid pipelines
Severe	75	>1/50	Expected structural damage to buildings and expected damage to rigid pipelines or possible damage to other pipelines

Source: Preene, M., *Proceedings of the Institution of Civil Engineers—Geotechnical Engineering*, 143(4), 177–190, 2000. With permission.

[a] The risk category is to be based on the more severe of the settlement or tilt criteria.
[b] Maximum settlement is based on the nearest edge of the structure to the groundwater control system.
[c] Tilt is based on rigid body rotation, assuming that all of the maximum settlement occurs as differential settlement across the width of the structure or across an element of the structure.

lowering projects (Preene 2000). Table 15.2 shows tentative values to be used in initial damage risk assessments for settlements caused by groundwater lowering.

15.4.6 Risk assessment of settlement damage from groundwater lowering

Once the settlement at various distances from the dewatering system has been estimated (using the equations outlined earlier and drawdown curves from Section 7.9), the values shown in Table 15.2 can be used to delineate risk zones. These are defined as areas where structures may experience particular levels of settlement and, hence, degrees of damage. The simplest form of risk zones assumes that soil conditions do not vary with distance. For radial flow, the risk zones will be a series of concentric circles centered on the groundwater lowering system (Figure 15.7a); for plane flow to pipeline trenches, the risk zones will be parallel lines on either side of the trench. The risk zones can be refined by including any known changes in ground conditions with distance. If settlements are significant, it is likely that compressible alluvial or postglacial soils are present. Geological mapping (from published data or from site investigation) can help determine the extent of these soils; damaging settlements are much less likely where these soils are absent. The application of geological mapping data will tend

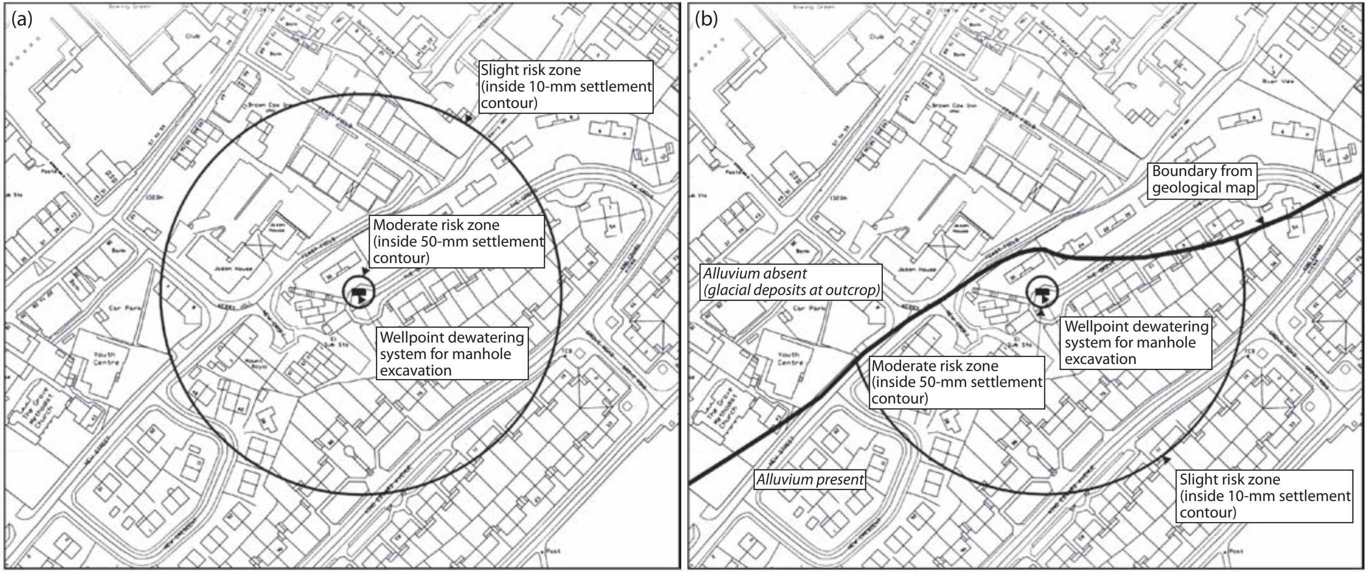

Figure 15.7 Settlement risk zones. (a) Idealized settlement risk zones for radial flow to a groundwater lowering system. Notes: (1) Predicted settlement at excavation is less than 75 mm; thus, no severe risk zone is generated. (2) No buildings lie in the moderate risk zone. (3) Numerous buildings lie within the slight risk zone. Building condition surveys should be considered for this zone. Any sensitive structures should be identified. (4) Settlement assessment assumes that the thickness of the alluvium is constant. Thickness is based on the borehole at the manhole location. (b) Settlement risk zones for radial flow to a groundwater lowering system based on variation in soil type from geological mapping. Notes: (1) Geological boundary taken from published geological mapping, confirmed by nearby boreholes. (2) Predicted settlement in an area where the alluvium is absent is less than 10 mm. (3) Settlement assessment assumes that the thickness of the alluvium is constant when present. Thickness is based on the borehole at the manhole location. (From Preene, M., *Proceedings of the Institution of Civil Engineers—Geotechnical Engineering*, 143(4), 177–190, 2000. With permission.)

Table 15.3 Suggested actions for settlement risk categories

Risk category	*Description of likely damage*	*Actions required*
Negligible	Superficial damage unlikely	None, except for any buildings identified as being sensitive, for which a detailed assessment should be made
Slight	Possible superficial damage; unlikely to have structural significance	Building condition survey to identify any preexisting cracks or distortions. Identify any buildings or pipelines that may be sensitive and carry out detailed assessment. Determine whether mitigation or avoidance measures are required locally
Moderate	Expected superficial damage and possible structural damage to buildings; possible damage to rigid pipelines	Building condition survey and structural assessment. Assess buried pipelines and services
Severe	Expected structural damage to buildings, and expected damage to rigid pipelines or possible damage to other pipelines	Determine whether anticipated damage is acceptable or whether mitigation or avoidance measures are required

Source: Preene, M., *Proceedings of the Institution of Civil Engineers—Geotechnical Engineering*, 143(4), 177–190, 2000. With permission.

to produce noncircular risk zones for radial flow (Figure 15.7b) and may reduce the extent of the zones and the number of structures at risk.

When the risk zones have been determined, and the number and type of structures at risk have been identified, appropriate action is required. Initial actions are summarized in Table 15.3. Additional, more detailed assessments may be needed for structures in the moderate and severe risk zones and for sensitive structures in the slight risk zone.

15.4.7 Mitigation and avoidance of settlement

Depending on the type of project and the number and nature of structures at risk, some (or perhaps all) of the anticipated damage may be deemed unacceptable. Table 15.4 lists settlement mitigation or avoidance measures, including the use of artificial recharge (see Section 11.9). Mitigation of environmental impacts is discussed in more detail in Section 15.9.

In addition to mitigation or avoidance, there is a third option, rarely considered explicitly—the acceptance of settlement. Powers et al. (2007) have suggested that, if damage risk is no more than slight, it may be more economical to accept third-party claims rather than deploying large-scale

Table 15.4 Measures to mitigate or avoid groundwater lowering-induced settlement damage

Mitigation of settlement	*Possible measures*
Protect individual structures	Prior to the works, underpin the foundations of some or all of the structures at risk
Reduce the number of structures at risk	Reduce drawdowns by reducing the depth of excavation below groundwater level
	Reduce the extent of the risk zones by minimizing the period of pumping
	Use cutoff walls to reduce external drawdowns
	Use an artificial recharge system to minimize external drawdowns
Avoidance of settlement	Relocate excavation away from vulnerable structures
	Redesign project to avoid excavation below groundwater level or excavate underwater
	Carry out excavation within a notionally impermeable cutoff structure[a]

Source: Preene, M., *Proceedings of the Institution of Civil Engineers—Geotechnical Engineering*, 143(4), 177–190, 2000. With permission.

[a] If a cutoff structure is used to avoid external drawdowns, it is essential that a groundwater monitoring regime is in place to allow any leaks in the cutoff to be identified before significant settlements can occur

mitigation measures. This might be quite a controversial approach on many projects and would present a public relations challenge but could be appropriate where relatively few structures were classified as being slightly at risk. A preconstruction building condition survey would be essential for this approach.

Powers (1985) suggests that a preconstruction building condition survey should include a photographic and narrative report on the interior and external condition of buildings. Particular attention should be paid to the condition of concrete foundations, structural connections, brickwork, and the condition of plasterwork or other architectural finishes that are particularly susceptible to cracking. Additionally, the condition of other structures must also be documented—examples include bridges, utility enclosures, and historic monuments. Paved surfaces (roads, pavements, and hardstandings) should also be examined, and their condition should be recorded. Ideally, the survey should be carried out by an independent organization (to avoid later charges of bias in the event of claims).

The legal position under United Kingdom law related to settlement damage resulting from the abstraction of groundwater is outlined in Section 17.4.

15.4.8 Impact on groundwater-dependent features

Groundwater can play an important role in supporting many surface water features. In many rivers, the flow is "supported" by groundwater (see Section 3.7). In other words, the rivers receive a contribution to their flow from groundwater (this groundwater-derived contribution is termed "baseflow"). Similarly, wetlands and ponds (which can be important ecological habitats) often exist because of the presence of groundwater flows or springs. Rivers, wetlands, and other phenomena whose stability and existence are partly controlled by the availability of groundwater are collectively termed "groundwater-dependent features."

The degradation of groundwater-dependent features (such as reduction in river flows and drying up of wetlands) caused by long-term abstraction for water supply is an issue that is widely recognized in water resource planning (Cunningham 2001). However, Acreman et al. (2000) noted that, in some cases, degradation of the aquatic environment believed to be linked to long-term groundwater abstraction may be due, at least in part, to other factors such as changes in land drainage, river channelization, and climate change.

Wetlands (areas of marsh, fen or peatland, or areas covered with shallow water, or poorly drained areas subject to intermittent flooding) can be particularly vulnerable to impacts from longer-term groundwater pumping. To assess each case, the interaction between the surface water and groundwater will need to be quantified. Some wetlands are directly supported by groundwater seepages, whereas other wetlands (if the soil is of low permeability) may receive little contribution from groundwater. Wetlands tend to vary with the seasons (and also from year to year); thus, the additional influence of groundwater pumping may or may not be significant in comparison. Merritt (1994) gives a thorough background to the creation and management of wetlands.

Although the impacts on groundwater-dependent features are commonly assessed during the development of water resource abstraction schemes, they are rarely considered during the development of groundwater control schemes. In practice, this probably reflects the fact that most dewatering schemes are of short duration and relative modest pumped flow rates and the potential for such impacts will be minimal on most sites. Nevertheless, there will be circumstances when these impacts should be assessed, particularly for long-term or "permanent" groundwater control systems.

Pumping of groundwater for groundwater control schemes can impact on groundwater-dependent features in one of two principal ways. First, the pumping may draw water from the feature, for example, by lowering groundwater levels so that water losses increase from the base of a pond or wetland. Second, pumping may intercept water that would otherwise have reached the feature. An example is where groundwater pumping reduces baseflow reaching a river, thereby reducing flow in the river. Assessment of water losses from groundwater-dependent features is not straightforward and can be complicated by uncertainties in the properties of semipermeable sediments in the base of rivers and ponds. Analytical methods for the assessment of impacts on groundwater-dependent features are discussed in the work of Kirk and Herbert (2002).

The degradation of groundwater-dependent features is most likely to be an issue for projects where

1. The proposed excavation is located close to the groundwater-dependent feature (Figure 15.8a), and groundwater is likely to be drawn from the feature even in the very short term.
2. The proposed dewatering system is likely to operate for a very long time (as is often the case with dewatering systems for open pit mines).

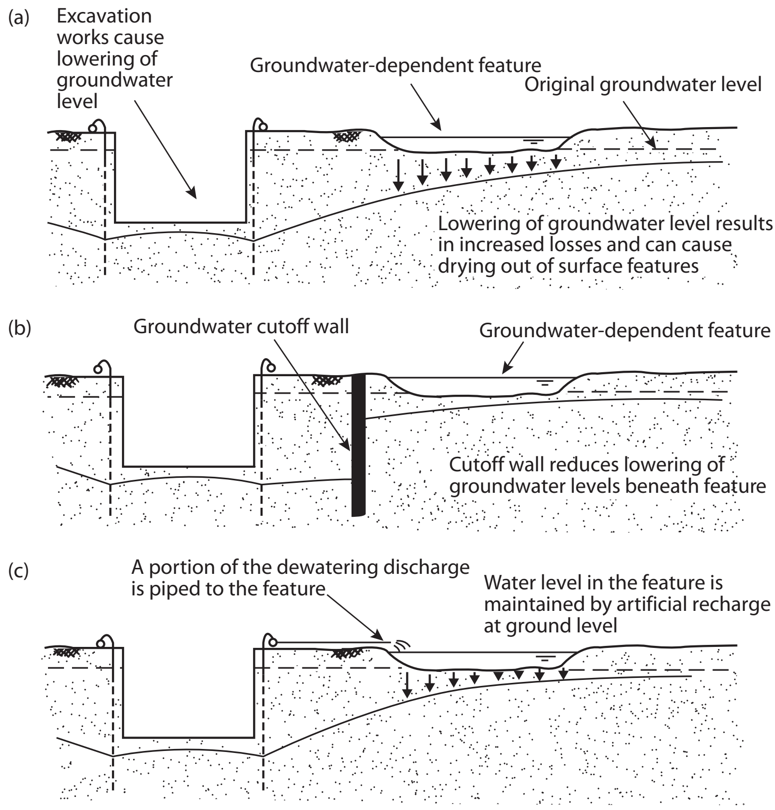

Figure 15.8 Depletion of groundwater-dependent features. (a) Depletion due to lowering of groundwater levels. (b) Cutoff wall used to mitigate impact. (c) Artificial recharge used to mitigate impact. (Redrawn from Preene, M., and Brassington, F.C., *Water and Environmental Management Journal*, 17(1), 59–64, 2003.)

These impacts should be considered for “permanent” dewatering systems (see Chapter 14).

3. The proposed dewatering system is operating at very high pumped flow rates, which have the potential for lowering groundwater levels over a very wide area and, therefore, may affect a larger number of sensitive sites.

When assessing the potential impact on groundwater-dependent features, specialist ecological and hydrological advice is likely to be required. It may be possible to reduce impacts by constructing a groundwater cut-off barrier between the dewatering system and the feature to be protected (Figure 15.8b). An alternative approach would be to use artificial recharge (see Section 11.9) of groundwater or surface water or to pipe a portion of the discharge water directly to the feature (Figure 15.8c). If the latter approach is adopted, the temperature, chemistry, and sediment content of the discharge water must be assessed to ensure that there will not be adverse reactions with the water and ecology of the feature; the risk of additional erosion must also be considered.

It is worth noting that, in many countries, including the United Kingdom, many wetlands and other sensitive groundwater-dependent features are “designated sites” under environmental regulations and, as such, are protected by law. Negotiations will have to be opened with the appropriate regulatory authorities when planning work near such sites. Similar protected environments exist in other parts of the world. This is an example where a desk study will be of immense value at the site investigation stage (see Section 6.4.1). In many countries, government bodies and environmental regulators maintain registers of designated sites. Identification of the presence of such a site during the desk study stage can allow further investigations to be made to prepare for future monitoring and mitigation requirements.

15.4.9 Effect on groundwater quality

Groundwater quality (i.e., the chemical composition of the water) varies naturally from place to place and aquifer to aquifer (see Section 3.9). In some cases, groundwater is almost pure enough to be potable with only minimal treatment in the form of chlorination to destroy any harmful bacteria. In other locations, the groundwater may contain considerable impurities, which could be naturally occurring or man-made. It is important to realize that pumping from groundwater lowering systems changes natural groundwater flow in aquifers and may cause existing contamination plumes or zones to migrate. Two of the most important cases to be considered are contaminated groundwater, often left over from industrial land use, or intrusion of saline water in coastal areas.

15.4.10 Movement of contaminated groundwater

The study of groundwater contamination is a major field in itself, and the reader is commended to texts such as the work of Fetter (1993) to obtain the full background on the subject. Contaminants interacting with groundwater flow exist in one of these three forms (or phases).

1. *Dissolved (or aqueous) phase*. A wide range of substances are soluble in water and, therefore, become part of the water itself, traveling with it.
2. *Nonaqueous phase*. This describes liquids that are immiscible with water. They may have densities less than water (light nonaqueous phase liquids [LNAPLs]) and will float on top of the water table (examples are petrol and diesel compounds). Dense nonaqueous phase liquids (DNAPLs) also exist, which are denser than water and tend to sink below the water table until they meet a low-permeability layer (examples include chlorinated hydrocarbons such as trichloroethylene). The nonaqueous phase is sometimes described as "free product," which means that it is a form of contamination existing separately (in a different phase) from the water.
3. *Vapor phase*. Volatile compounds in the contaminant can move in gaseous form in the unsaturated zone above the water table.

Some contaminants (e.g., hydrocarbons such as petroleum products) may create all three phases when they reach the groundwater.

Contamination may be caused by a variety of land uses:

1. Industrial processes—mainly from spillages or leakages from stored materials. Sites of concern include not only manufacturing and chemical production sites but also vehicle storage areas, airports, and other locations where fuel and detergents are used.
2. Landfilling and waste disposal—both from official and unofficial disposals.
3. Agricultural practices—such as fertilizer or pesticide use.
4. Urban use—including leaking sewers and fuel spills.

The absence of the active use of a site does not provide assurance that the site is uncontaminated. A legacy of contamination may exist in the ground and groundwater for years or decades after the pollution has stopped. A guide to the types of pollution that can be expected from former industrial sites can be found in the Construction Industry Research and Information Association (CIRIA) reports on contaminated land (Harris et al. 1995). A desk study (see Chapter 6) should investigate former uses of a site to determine the risk of contaminants being present at problematic levels.

As described in Chapter 3, groundwater is constantly in motion, and where contamination exists, that will tend to move too, gradually forming a plume stretching away from the original source of contamination. The rate and direction of movement of contamination depends on many factors, including hydraulic gradients, the geological structure of the aquifer, the nature of the contaminant, and any chemical changes in the contaminant with time. Detailed consideration of these factors is beyond the scope of this book, and the references cited earlier are recommended for further study. However, it is vital that anyone designing or carrying out a groundwater lowering exercise understands that pumping may change considerably the existing groundwater gradients and velocities, affecting both the magnitude (generally increasing flow velocities) and direction. This means groundwater lowering can cause the extent of a contamination plume to change, perhaps much more rapidly than previously. If the movement of contamination is of real concern, a thorough site investigation followed by the development of a groundwater flow and contaminant transport model is essential, and specialist advice should be obtained at an early stage.

When planning groundwater lowering on or near a contaminated site, there are two important issues to be addressed, in addition to the dewatering design itself:

1. How can the influence of groundwater lowering on the contamination plume be minimized or controlled? The use of physical cutoff barriers (see Chapter 12) to hydraulically separate the groundwater lowering system from adjacent contaminated sites is a method often used (see Section 11.10).
2. How can the discharge be disposed of? The water pumped from the wells may contain problematic levels of contamination, preventing direct discharge to watercourses or sewers. On occasion, it has been necessary to establish a temporary water treatment plant on the site to clean up the discharge water quality (see Section 11.10).

Methods based on groundwater lowering technology can be used to help clean up sites. Pumping of groundwater and treatment of discharge prior to disposal, with the aim of reducing contamination levels, is known as the "pump and treat" method. This is a specialist method, and its effectiveness should be compared to other competing cleanup techniques (Holden et al. 1998). Further details are given in Section 11.10.

15.4.11 Saline intrusion

Saline intrusion describes the way more mineralized water is drawn into freshwater aquifers under the influence of groundwater pumping. This is a particular problem where large volumes of groundwater are abstracted for potable supply, because if saline water reaches the well, it may have to be abandoned. Saline intrusion principally affects coastal aquifers, but saline water can sometimes be found in inland aquifers, where the water has become highly mineralized at depth.

Saline intrusion is a complex process affected by aquifer permeability, rate of recharge, natural groundwater gradients, and the effect of any existing pumping wells. Any significant groundwater lowering operations will affect the boundary between fresh and saline water. If saline water is drawn to the groundwater lowering system, that may not be a problem in itself (provided that the water can be disposed of), but any saline water drawn toward nearby supply wells is of much greater concern. The risk of saline intrusion may need to be investigated using hydrogeological modeling to assess the effect on local and regional water resources; the reader is referred to hydrogeological texts such as the work of Younger (2007) for further details.

15.4.12 Effect on groundwater borehole or spring supplies

This book mainly deals with groundwater as a problem, needing to be controlled to allow construction excavations to proceed, but as highlighted at the start of this chapter, groundwater is also a resource used by many. Groundwater is obtained from wells and springs as part of public potable water supplies and for private supplies for domestic dwellings and industrial users such as breweries and paper mills. If temporary works groundwater lowering is carried out in the vicinity of existing well or spring abstractions, there is a risk that the abstractions will be "derogated." In other words, it will be harder for the user to abstract water, and the source may even dry up completely in extreme cases (Figure 15.9). The interaction between groundwater supplies and civil engineering works is discussed further by Brassington (1986).

Occasionally, water quality from a spring or well source may deteriorate as a result of changes in the groundwater flow direction. The effect is often temporary and may cease soon after the end of temporary works pumping but can cause considerable inconvenience and cost to groundwater users. The legal issues must also be considered, because in England and Wales, licensed groundwater abstractors have a legal right to continue to obtain water (see Section 17.4).

When groundwater lowering is carried out for a construction project, the primary effect on nearby abstractions is likely to be a general lowering of water levels, which will affect operating water levels in existing wells (see Figure 15.17), with a corresponding reduction in output. The magnitude of the reduction in output will depend on several factors:

1. Aquifer characteristics, including permeability and storage coefficient
2. Distance between groundwater lowering wells and supply wells, and their location in relation to any existing hydraulic gradients in the aquifer

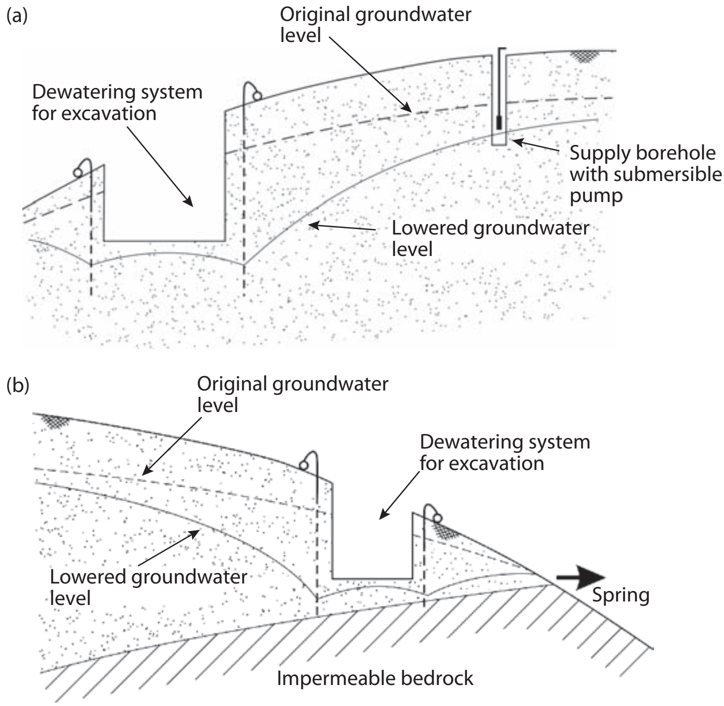

Figure 15.9 Derogation of groundwater sources. (a) Impact on borehole. Dewatering system lowers groundwater level at the water supply borehole. (b) Impact on the spring. Dewatering system reduces flow from the spring. (After Preene, M., and Brassington, F.C., *Water and Environmental Management Journal*, 17(1), 59–64, 2003.)

3. The dewatering pumping rate and period of pumping (low flow rate and short-duration pumping systems will have less of an effect on supply wells)
4. The depth, design, and condition of the supply wells

Any rational assessment of the effect of groundwater lowering on supply wells will require some form of conceptual groundwater model to be developed. This could then be used as the basis for a numerical model, or an initial assessment could be made using the methods in Chapter 7, treating the groundwater lowering system as an equivalent well. The assessment of potential impacts is discussed in Section 15.9.

Permanent dewatering systems (see Chapter 14) can also have an impact on nearby groundwater sources. Impacts may be caused not only by systems that are actively pumped by wells but also by projects where linear engineered features (such as road or rail cuttings) are drained by gravity (Figure 15.10).

If the estimated effects on the supply wells are small, this may be deemed acceptable with no further mitigation measures. However, if the effects are more severe, Powers (1985) suggests the following mitigation measures:

1. If only a few low-volume users are affected and the dewatering period is short, the lost supply might be replaced by a temporary tanker supply.
2. If the supply well is deep but with the pump set at a fairly high level, it may be possible to install higher head pumps at a greater depth in the supply well. This would allow abstraction to continue even with the additional drawdown generated by groundwater lowering.
3. If the supply wells are shallow, it may be necessary to deepen the wells, perhaps into another aquifer. Alternatively, for small-diameter shallow wells, it may be more economical to simply drill a new, deeper well.
4. A portion of the dewatering discharge may be piped to the affected user by a temporary pipeline. Point-of-use treatment, which is dependent on water quality, may need to be provided to ensure that the water is suitable for use.
5. Public water mains may be extended into the affected area, giving a permanent benefit for the money spent.

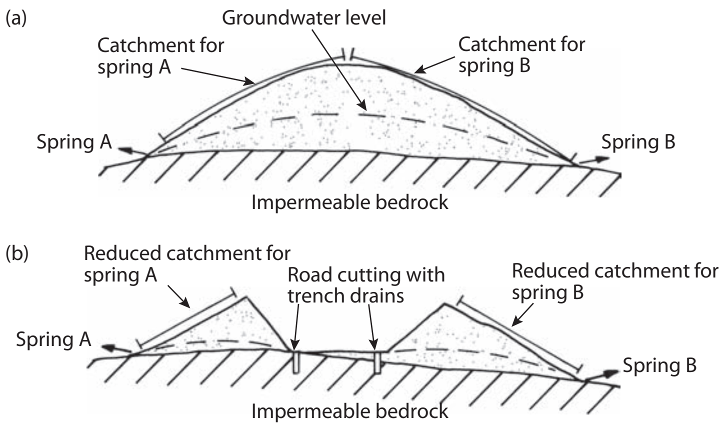

Figure 15.10 Groundwater abstraction from linear construction projects. (a) Flow to springs prior to construction. (b) Reduced flow to springs following construction. (Redrawn from Preene, M., and Brassington, F.C., *Water and Environmental Management Journal*, 17(1), 59–64, 2003.)

Some of these measures have huge cost, time, and public relation implications and clearly need to be compared with the alternative of constructing the project without groundwater lowering or even of relocating the project away from the supply wells.

A desk study (see Section 6.4.1) will be valuable in identifying the presence of any vulnerable water abstractions (well or spring sources) that may be impacted by a proposed groundwater control scheme. National and regional governmental bodies and environmental regulators often hold records of the location of water sources. Larger sources may have source protection zones (SPZs) delineated around them, within which engineering works require special permission from the regulators.

If the effects on nearby groundwater abstractions are of real concern, it is essential that they are addressed early in the planning of a project, because it is unrealistic to expect the contractor to bear all the costs and risks of some of these measures. The project client will have to face up to the potential need for some of these measures and perhaps allow for them when negotiating with landowners for wayleaves. The mitigation measures might be included at the very start of site works as part of the enabling works. Alternatively, the supply wells may be monitored during the works, with a contingency in place that the mitigation measures will be applied if the well is affected beyond a certain predefined level.

15.4.13 Other effects

Occasionally other, less common, side effects may be of concern.

15.4.13.1 Damage to timber piles

It is widely recognized that timber piles supporting older structures may be detrimentally affected by the drawdown of water levels. This is a particular issue in Scandinavia, where buildings founded on timber piles are commonplace (Peek and Willeitner 1981). In cities such as Copenhagen, this is such an important issue such that it has led to the introduction of location regulations that prohibit significant lowering of groundwater levels in specified areas. This has influenced the groundwater control techniques used on several major infrastructure projects in Copenhagen, leading to the widespread use of artificial recharge systems (Bock and Markussen 2007).

Powers (1985) states that the damage to timber piles and foundations may result from fungi present in the timber thriving in an aerobic environment created if water levels are drawn down, exposing the tops of the piles to air. However, Powers also states that the most severe cases of aerobic attack have been when piles were exposed in excavations and that the observed decay due to drawdown has been less severe. This is probably because the oxygen supply to the timber surface is not increased substantially when the piles are in dense or fine-grained soils.

Nevertheless, a sensible approach is to proceed cautiously when working in areas when older structures are founded on timber piles. Even if aerobic attack does not compromise pile stiffness, soil consolidation and pile downdrag due to negative skin friction should be considered.

15.4.13.2 Vegetation

It is rare for groundwater lowering systems to have a noticeable effect on vegetation. This is mainly due to the short-term nature of pumping and the fact that plants generally draw their water from immediately below the surface, above the water table. This zone is much more likely to be affected by changes in precipitation and infiltration than by deeper pumping. Longer term pumping (for certain types of quarries or open pit mines) may need to consider this issue further, and the services of an experienced ecologist can be very useful in that regard.

15.4.13.3 Impact on archaeological remains

The continued in situ preservation of archaeological remains may also be dependent on stable groundwater levels, and there have been cases of degradation associated with large-scale dewatering works (French and Taylor 1985). Numerical modeling has been used to assess groundwater lowering beneath areas of archaeological interest (Garrick et al. 2010).

15.5 IMPACTS FROM GROUNDWATER PATHWAYS

One potential impact that is sometimes overlooked is the potential for groundwater to flow along permeable pathways created by wells, piles, excavations, or even a structure itself. Flow along these pathways can affect groundwater quality or the quantity of water available to groundwater sources.

Some of the groundwater pathways may be temporary (such as investigation and dewatering boreholes) and can be sealed on completion. Other pathways are formed by parts of the structure or works and may exist in perpetuity. Examples of permanent pathways include the granular bedding of pipelines (which may allow horizontal flow) or some types of piling or ground improvement processes (which can form vertical pathways). Open excavations such as road or rail cuttings may themselves form vertical pathways.

It is now recognized that the vulnerability of aquifers to pollution resulting from surface sources (e.g., surface runoff and fuel spills) will depend on the nature of the aquifer and any overlying strata. For example, a high-permeability unconfined aquifer will be much more vulnerable to contamination than a deep confined aquifer overlain by a thick clay layer, which can act as a barrier to pollution. In the United Kingdom, aquifer vulnerability maps have been produced for many areas (Palmer and Lewis 1998).

15.5.1 Vertical pathways via boreholes and wells

The installation of wells and boreholes may puncture low-permeability layers, increasing the risk of surface pollution finding its way down into the aquifer. This is of particular concern if the near-surface strata have been contaminated by historic or ongoing polluting industries.

Changes in groundwater quality may result if pathways are formed between different aquifer units. For example, poorly sealed investigation boreholes could allow mixing of fresh and more saline water in aquifers where groundwater quality is stratified, or polluted groundwater at shallow depth may flow into deeper aquifers (Figure 15.11).

If changes in aquifer vulnerability are of concern, a number of mitigation measures should be considered:

1. The well design should include appropriate grout seals to prevent vertical seepage around the outside of the well casing.
2. The well casing should stand sufficiently proud of ground level to prevent surface waters from being able to pass into the well casing (and then into the aquifer) in the event of localized flooding around the well.

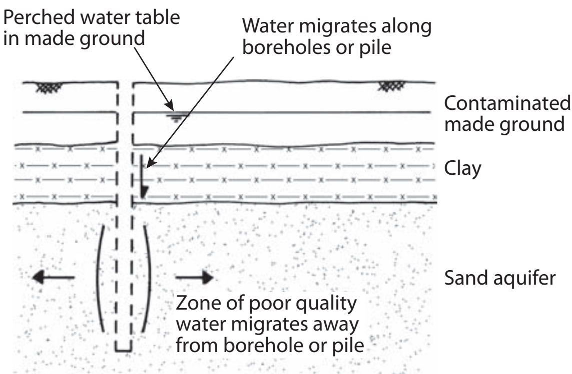

Figure 15.11 Vertical pathways for groundwater. Borehole or pile punctures clay aquiclude and may allow water from contaminated near-surface layers to seep downwards and contaminate lower aquifer. (Redrawn from Preene, M., and Brassington, F.C., *Water and Environmental Management Journal*, 17(1), 59–64, 2003.)

3. The top of the well casing should be capped when not in use and sealed around the pumping equipment when in use. This will reduce the chances of noxious substances being dropped or poured down the well, either maliciously or by accident.
4. The wells should be adequately capped or sealed on completion (see Section 16.8).
5. Where flowing artesian conditions (see Section 3.4.2) exist in the aquifer, the design of dewatering should include suitable grout seals and headworks to allow the well to be sealed when not in use to prevent uncontrolled artesian discharges.

15.5.2 Vertical pathways via excavations and structures

In addition to the drilling of wells and boreholes, excavations or structures also have the potential for creating vertical pathways. It is rare that sufficient groundwater level monitoring is carried out to identify changes in groundwater levels due to the installation of piles or substructures. Ervin and Morgan (2001) recorded a temporary reduction in piezometric level of up to 1.5 m (over a period of 3 months) in a low-permeability aquitard underlain by a more permeable gravel aquifer. They attributed this change to the installation of bored piles into the gravel through the aquitard. Piezometric levels were recovered following the end of the piling works, implying that the impact was associated with the pile construction and not the long-term presence of the piles.

Where aquifer conditions are sensitive (e.g., in the inner catchments around public water supply wells), deep structures such as shafts or basements should be designed to limit the potential for creation of vertical flow paths. This can be done by using raft foundations in preference to piles that would puncture low-permeability aquitard layers.

If piling or ground improvement methods have to be used, methods should minimize the formation of vertical flow paths. Guidance is given in the work of Westcott et al. (2001).

15.5.3 Horizontal pathways

Linear horizontal underground structures such as pipelines may act as horizontal conduits for groundwater flow. A classic example is where a sewer pipeline is laid on granular bedding material. The bedding is likely to be of high permeability and will allow preferential flow of groundwater along its length. This can divert flow away from nearby groundwater sources such as springs or wells (Figure 15.12). Problems can also occur when dewatering near existing structures or pipelines that act as pathways. Dewatering

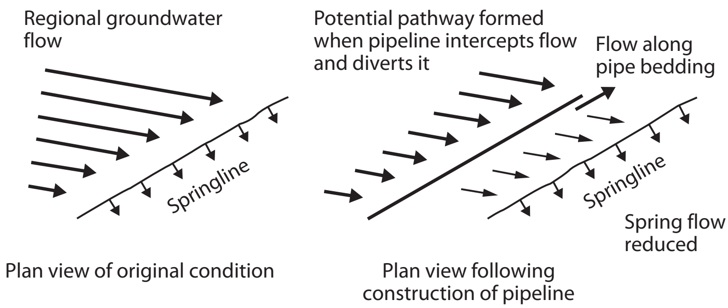

Figure 15.12 Horizontal pathways for groundwater flow. Groundwater flow is diverted along the permeable bedding of the pipeline, reducing the flow to the springs. (Redrawn from Preene, M., and Brassington, F.C., *Water and Environmental Management Journal*, 17(1), 59–64, 2003.)

flow rates in these areas can be much greater than anticipated, as water is drawn toward the pumping wells along the permeable bedding around the structure or pipeline.

Potential mitigation measures that address the issues associated with horizontal pathways include

1. Horizontal structures such as pipelines should be constructed with low-permeability barriers or antiseepage collars (known as "stanks") at regular intervals along their route. This will reduce the potential for horizontal groundwater flow.
2. Where dewatering is planned near an existing structure that has the potential to act as a horizontal pathway (e.g., where permeable bedding may be present), grouting is sometimes used to seal the pathway and reduce dewatering flow rates.

15.6 IMPACTS FROM GROUNDWATER BARRIERS

Many groundwater control schemes use low-permeability cutoff walls that act as barriers to exclude groundwater from excavations. The range of techniques used to form cutoff walls is described in Chapter 12. In many cases, these cutoff walls are effectively permanent, remain in place following the end of the construction period, and may interrupt horizontal groundwater flow, causing a damming effect (Figure 15.13a). These effects may not be significant, unless large structures fully penetrate significant aquifer horizons. It is rare that sufficient groundwater monitoring is carried out to allow these effects to be quantified; Barton (1995) recorded groundwater level rises of 0.2–0.8 m upstream of a structure that fully penetrated a valley gravel aquifer.

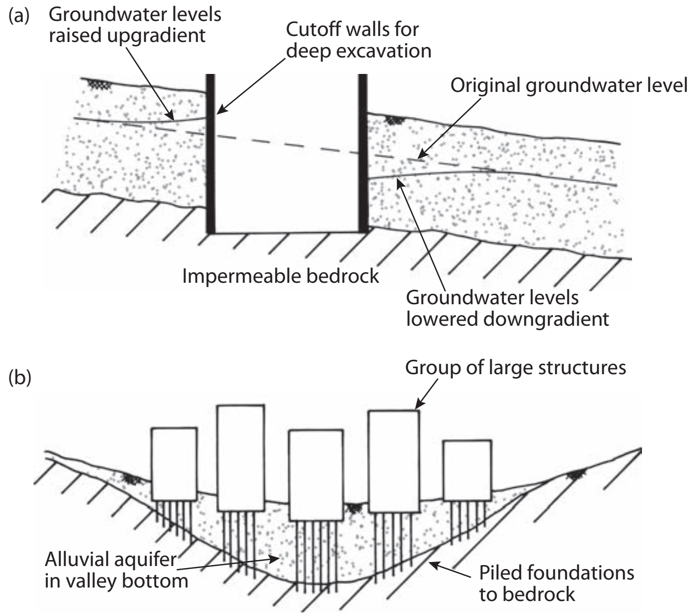

Figure 15.13 Barriers to groundwater created by cutoff walls and piles. (a) Cutoff walls. (b) Groups of piles. (After Preene, M., and Brassington, F.C., *Water and Environmental Management Journal*, 17(1), 59–64, 2003.)

In addition to cutoff walls installed with the objective of blocking groundwater flow, groundwater barriers may inadvertently be formed where extensive heavy-duty foundations are installed into aquifers that are shallow or of limited thickness (Figure 15.13b).

In reality, the impact of groundwater barriers will be modest for most engineering structures. The exception is where very long linear structures (such as metro stations or cuttings for roads or railways) are contained within low-permeability walls. If such structures are located across the direction of natural groundwater flow, then groundwater flow will be diverted around the sides of the structure. This can reduce the supply to nearby groundwater sources or cause flooding of adjacent basements upstream of the structure.

Where impacts are a potential concern, it may be appropriate to use numerical groundwater modeling to assess changes in groundwater level. If impacts are assessed to be significant, then consideration should be given to modifying the cutoff wall or piled foundation design to limit the depth of piles or cutoff walls or to use cutoff walls that are temporary in nature (such as artificial ground freezing or steel sheet piles removed at the end of construction).

An additional impact occasionally associated with groundwater barriers is groundwater contamination derived from the materials used in the barrier. This is a potential issue for grout barriers formed from chemical grouts (see Section 12.9), where the groundwater chemistry may have the potential for leaching polluting chemicals from the grout into the groundwater.

15.7 IMPACTS FROM DISCHARGE FLOWS TO THE GROUNDWATER ENVIRONMENT

The construction activities associated with civil engineering excavations can create the potential for discharges to groundwaters, with the consequent risk of pollution and degradation of groundwater quality. The main sources of potentially polluting discharges are leakages and spills of fuels and lubricants from plant and vehicles, runoff from operations such as concrete placement or grouting, and runoff of turbid surface water as a result of topsoil removal and excavation. Normally, the risk of polluting discharges can be reduced by the adoption of good practice (for example, see the work of Murnane et al. 2006) and guidance from the environmental regulators for the site locality.

The risk of pollution is increased if pathways for groundwater flow (see Section 15.5) are associated with the works (Figure 15.14). Oftentimes, open excavations form a ready pathway for inadvertent discharges to groundwater. Good site practice should include prohibiting refuelling of plant (and storage of fuels) in or near excavations. Surface water drainage (see Section 5.2) should be arranged to reduce the risk of spills or runoff entering the excavation.

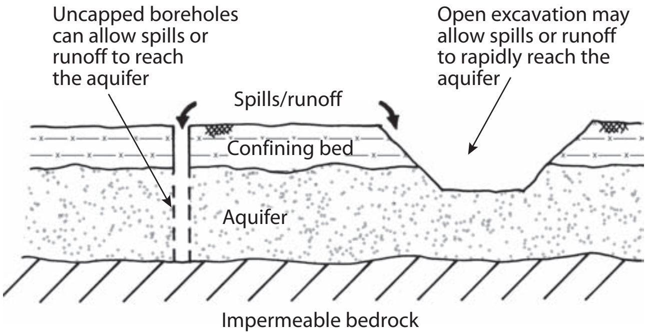

Figure 15.14 Potentially polluting discharges to groundwater. (Redrawn from Preene, M., and Brassington, F.C., *Water and Environmental Management Journal*, 17(1), 59–64, 2003.)

Structures with deep basements or belowground spaces may also provide potential for discharges to groundwater in the longer term. If the structures are not watertight and penetrate confining beds over aquifers, leaks, spillages, or surface water flooding may percolate more freely into the groundwater. Individually, such leakages may be small, but their combined effect may lead to significant groundwater contamination.

The use of artificial recharge (see Section 11.9) also creates the risk of groundwater pollution if the reinjected water becomes contaminated as a result of its pumping and transfer through the recharge system. Great care must be taken to reduce the risk of recharge water becoming contaminated.

There have been rare occasions when the release of chemicals from processes used in ground engineering has also been a focus of concern. This may be relevant when works are carried out in aquifers that are used for drinking water supply or are very sensitive for ecological reasons. This was a particular concern during the 2000s on the Copenhagen Metro Project, which involved extensive tunnel construction. Raben-Levetzau et al. (2004) describe how chemical grouts were prohibited and numerical groundwater modeling studies were carried out to assess whether migration of products used in tunnel construction (soil conditioning products, lubrication and sealing projects, and dispersants) might affect surrounding water quality.

On another project, a tunnel was being driven by a tunnel boring machine (TBM) through a fissured carbonate rock aquifer relatively close to a high-capacity public drinking water supply borehole. There was a concern that the cutting action of the TBM might create loose particles that could be drawn to the pumping borehole along enlarged fissures believed to be feeding the borehole. This could create turbidity in the pumped water, making it unsuitable for drinking water use. The strategy adopted was to take the water supply borehole out of service while the TBM traversed the area. When the borehole was restarted after completion or tunneling, it was pumped to waste for an extended period and closely monitored for turbidity to ensure that any turbid water did not find its way into the supply.

15.8 IMPACTS FROM DISCHARGE FLOWS TO THE SURFACE WATER ENVIRONMENT

Any system that lowers groundwater by pumping will produce a discharge flow of water that must be disposed of. The most commonly used routes for the disposal of discharge water from dewatering systems include

1. Discharge to surface waters (e.g., river, watercourse, lake, and sea)
2. Artificial recharge (see Section 11.9) to groundwater (e.g., via recharge wells or recharge trenches)
3. Discharge to an existing sewerage network

Legal permissions necessary for the discharge of groundwater are outlined in Section 17.4.

Poorly managed discharges may have adverse impacts on the environment. This section describes good practice for discharges to the surface water environment.

15.8.1 Erosion caused by discharge flows

Poorly located discharges can cause erosion of riverbanks or watercourses if the flow is concentrated in one location, particularly if the flow rate is large. A scour hollow will form under the discharge point, possibly undermining the bank. Problems may be created downstream, as the scoured material is redeposited, blocking or changing the flow in the watercourse.

This problem can be avoided by designing the discharge system to reduce the potential for scour. Materials such as gabion baskets, stone or geotextile mattresses, or even straw bales can be placed at the discharge point to dissipate the energy of the water before it passes into the watercourse proper.

15.8.2 Suspended solids

Suspended solids in the form of clay-, silt-, and, occasionally, sand-sized particles are a common problem resulting from dewatering discharges. Silt discharges are a highly visible aesthetic problem (Figure 15.15), but silt also harms aquatic plant, fish, and insect life and can build up in watercourses, blocking flow.

The suspended solids in sediment-laden discharges can be difficult to deal with economically; thus, the best approach is to tackle the problem at source and avoid silt being drawn into the discharge water. This requires appropriately designed and installed filters to be included in the system design. This is the norm for the wellpoint, deep well, and ejector methods, and such systems do not commonly produce discharges with high sediment loads, except for a short period during initial well development.

Most problems with suspended solids in discharge water arise from sump pumping operations (see Section 8.8). In many cases, adequate filters are not installed around the sumps, and fine particles can be drawn out of the soil and entrained in the discharge water. Where sump pumping is carried out in soils containing a significant proportion of fine particles, the discharge will need to be treated to reduce any suspended solids to acceptable levels prior to discharge. If treatment is not possible, a change of dewatering method (e.g., by using wellpoints with adequate filters) should be considered.

Figure 15.15 Silt plume in a watercourse resulting from the discharge of water from poorly controlled sump pumping. (Courtesy of T.O.L. Roberts.)

The most common form of treatment for suspended solids is by passing through settlement tanks or lagoons. In essence, settlement tanks provide an environment (a tank or lagoon) of relatively still water through which the discharge flow passes very slowly and, hence, has a long “retention time.” This allows solid particles to settle out of the water and be deposited in the base. The water, now with a reduced load of suspended solids, is discharged from the outlet side of the tank, typically via a weir or high-level outlet pipe. The solids will be retained in the base of the tank or lagoon. Periodically, the tank may need to be drained, and the solids should be removed.

The settling rate of a solid particle is controlled by Stokes’ law—the smaller the particle, the slower its rate of settling. Hence, the larger the tank, the longer the retention time, allowing more time for smaller particles to settle out. In order to settle out silt- and clay-sized particles by conventional means, very large tanks or lagoons may be needed.

The efficiency of settlement can be improved by chemical treatment, whereby coagulants and flocculants are added to encourage groups of particles to coalesce together and form larger “flocs” of particles, which will settle much more quickly (Smethurst 1988). Chemical treatment of water is

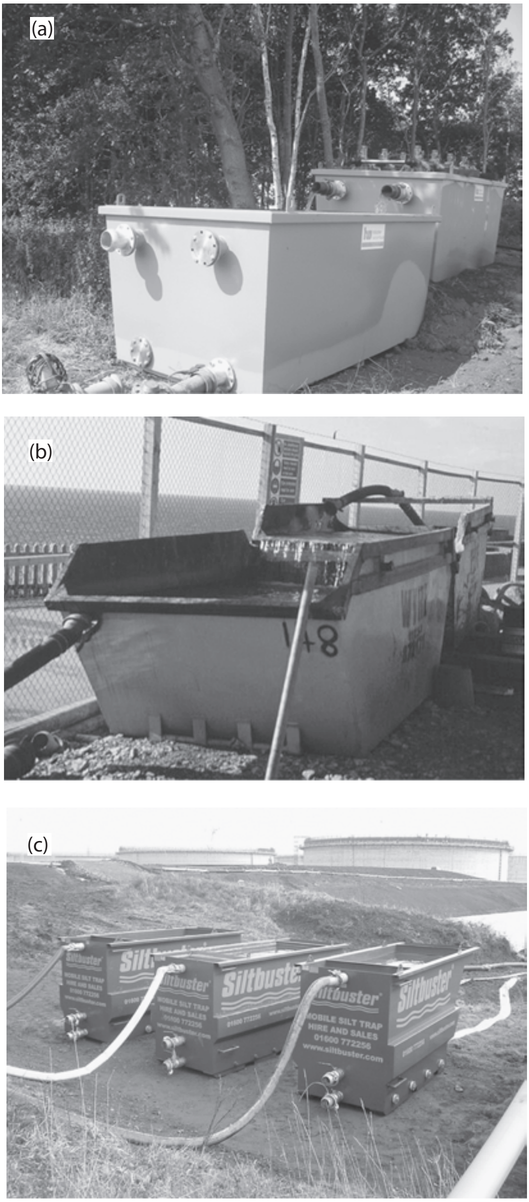

Figure 15.16 Treatment of discharges by settlement. (a) Modular settlement tanks. (Courtesy of Hölscher Wasserbau GmbH, Haren, Germany.) (b) Small settlement tanks improvised from waste skips. (c) Specialist lamella settlement tank. (Courtesy of Siltbuster Ltd., Monmouth, U.K.).

Figure 15.16 (Continued) (d) Large settlement lagoons.

a complex process and specialist advice should be obtained if this approach is being considered.

The most common form of settlement tanks used on dewatering projects are small portable steel tanks (typically 3 m × 1.5 m in plan and up to 1.5 m deep). Purpose-built tanks are available (Figure 15.16a), or improvised tanks can be made from waste skips available on the site (Figure 15.16b). Such tanks can be effective in removing sand-sized particles, but silt- and clay-sized particles settle slowly and will pass through these small tanks.

In recent years, some innovative approaches have been developed to allow more effective settlement of solids via small tanks. One example is the Siltbuster® unit (Figure 15.16c) based on the principle of lamella plate clarification, a process widely used in the treatment of drinking water. In this approach, the dirty water is passed upward between inclined parallel plates that are separated by a small gap. Solid particles settle onto the inclined plates and fall to the base of the tank. The lamella plate approach allows a small tank, comparable in plan size to a conventional steel settlement tank to have the settling capacity of a lagoon several times its size.

Large lagoons can be used to provide sufficient retention time for silt- and clay-sized particles to settle out. A typical lagoon is shown in Figure 15.16d and consists of an earthwork bund or pit with some form of waterproof lining on the base and sides; edge protection is necessary to reduce hazards to personnel. It may be necessary to operate two or more lagoons in parallel. When one lagoon is full, the other receives the discharge, allowing more time for settlement in the first. Water from each lagoon is decanted and disposed of in turn; sediment needs to be removed periodically from the lagoons.

15.8.3 Oil and petroleum products

Oil- and petroleum-based products may find their way into dewatering discharges as a result of spills or leaks from plant or fuel storage areas. This is a particular risk with sump pumping, because any spills or leaks in the base of the excavation will be carried to the pumps by surface water. Petroleum products may also occur in discharges if contaminated groundwater is pumped.

Petroleum products are generally of lower density than water and do not mix well with it. They are known as LNAPLs and will tend to appear as floating films or layers on top of ponds or tanks of water. The petroleum products must be separated from the discharge before disposal. This can be accomplished by passing the discharge through proprietary "petrol interceptors" or "phase separators." The water is then discharged as normal, and the product (which collects in the interceptor) is disposed of (e.g., to a waste oil company) at appropriate intervals. Very thin oil layers may be removed by the use of floating skimmer pumps or sorbent booms or pillows placed in discharge tanks or lagoons.

15.8.4 Contaminated groundwater

If groundwater is pumped from a contaminated site or from adjacent to such a site, the resulting dewatering discharge may itself be contaminated. Unless the flow is discharged to a sewage treatment works capable of dealing with the contaminants, some form of on-site treatment will be required prior to discharge.

A wide range of treatment methods is available. Oftentimes, a given contaminant could be treated by several quite different methods; the choice of method will depend on the concentration of contaminants, the discharge flow rate, the duration of pumping, and the availability of treatment equipment and technologies. Some of the available technologies are described in Section 11.10.

The scale of treatments used in practice varies greatly. Occasionally, on very heavily contaminated sites with low pumped flow rates, the discharge may be pumped directly to special road tankers, for off-site disposal at a licensed facility. In other cases, on-site treatment may be feasible. This ranges from simple dosing with caustic soda for pH adjustment of acidic groundwater to the construction of modular treatment plants on heavily contaminated sites (see Section 11.10).

A program of chemical testing will be needed to monitor discharge water quality, and the environmental authorities (see Section 17.4) will need to be kept fully informed. Oftentimes, the targets for treatment of the discharge will be set by the environmental regulators based on a consideration of the environmental impact of the discharge.

15.9 ASSESSMENT OF POTENTIAL ENVIRONMENTAL IMPACTS

For any groundwater control project of any substantial size, it will be necessary to assess the potential environmental impacts. It may also be necessary to consider potential impacts for smaller groundwater control systems in potentially sensitive locations (e.g., on construction sites near water supply wells or groundwater-dependent features).

There are four main stages in the assessment and management of potential environmental impacts from groundwater control activities. These are identification, prediction, mitigation, and monitoring.

15.9.1 Identification of potential environmental impacts

The first stage is to assess, in principle, the potential for significant impacts to occur. The key element of this stage is to develop an appropriate conceptual model of the groundwater conditions (see Section 7.4). The conceptual model must include factors related to the ground and groundwater conditions, to the groundwater control technique to be used, and to the presence and location of environmental features that may be impacted by the range of impacts listed in Section 15.3.

A desk study (see Section 6.4.1) can be of great value at this stage to help identify the existence of environmental features that have the potential to be impacted. Where features such as wetlands and private water supply wells are identified, it may be appropriate to carry out a site inspection visit of the feature to gather further information.

At this stage, the existence of sources of background or baseline monitoring data (see Section 15.9.4) to determine existing groundwater levels is also worth investigating.

This initial stage of assessment should have an additional objective of assessing the level of uncertainty in the information relating to environmental impacts. The identified uncertainties should be used to help plan future monitoring and/or investigations to fill any data gaps.

15.9.2 Prediction of potential environmental impacts

Based on the previously identified potential for impacts, it is necessary to make some kind of assessment of the magnitude of the impacts to determine whether any mitigation may be necessary.

The choice of methods used to quantify the impacts will depend on the conceptual model and on the scale and type of impact anticipated. For complex projects, numerical groundwater modeling (see Section 7.10) may be used, whereas for many projects, it may be appropriate to use conventional analytical design equations (see Chapter 7) to assess drawdown impacts. It is important to realize that, oftentimes, the objective of calculations or modeling is not necessarily to make precise predictions of the magnitude of impacts. Instead, the objective of the predictions is to aid decision making by determining whether the impacts are likely to be significant enough to require mitigation.

15.9.3 Mitigation of potential environmental impacts

The selection of any mitigation measures should be based on the conceptual model and the predicted impacts.

Three principal types of mitigation can be applied to pumped dewatering schemes:

1. *Return of pumped water to the environment*. Many of the potential impacts derive directly from the removal of water from the aquifer system, for example, ground settlement, impacts on groundwater-dependent features, and loss of yield of water supply wells. These impacts can be mitigated by returning the water back to the area of impact. This could include artificial recharge to the aquifer (see Section 11.9) or discharge of water to affected wetlands or rivers.
2. *Prevention of the removal of groundwater from sensitive areas*. This can involve the use of groundwater cutoff barriers (see Chapter 12) to prevent groundwater flow in unfavorable directions, to prevent seepage from rivers and wetlands, and so on. Additionally, appropriate design of dewatering wells and monitoring wells with screens and grout seals at appropriate levels can help control vertical flow of groundwater, for example, to prevent excessive drawdowns in shallow strata when dewatering is carried out in deeper layers.
3. *Avoidance or reduction of groundwater lowering*. The ultimate mitigation is to consider changing the scope of the overall project to reduce or avoid the need for groundwater control. Most commonly, this would involve the redesign of the project to reduce the depth of excavation or the relocation of the project to increase the distance to any identified environmentally sensitive features. Obviously, such a mitigation measure is only feasible in the very early planning stages of a project before location and design are finalized, but this approach is sometimes used during the route planning stage for tunnel projects or linear infrastructure projects such as new roads or railways. On tunnel projects, the location of shafts (which may require dewatering) may be deliberately located in areas distant from potentially sensitive groundwater-dependent features.

Mitigation measures for specific categories of impacts are described earlier in this chapter, alongside the description of the relevant impacts.

15.9.4 Monitoring of potential environmental impacts

Monitoring of the groundwater regime around a site is an essential part of identifying and mitigating potential impacts. Typical parameters monitored include

1. Groundwater levels in wells and boreholes
2. Surface water levels in wetlands, streams, etc.
3. Flow from springs and in associated watercourses
4. Flow in rivers where groundwater baseflow is anticipated to be a key component of flow
5. Water quality parameters at springs or boreholes, including the use of geophysical fluid logging in boreholes with stratified water quality

Location of monitoring points should be controlled by the conceptual model of the anticipated impacts. It is clear that monitoring points must be located in areas and aquifer units where impacts are expected. However, it is also prudent to carry out monitoring in aquifer units where no impacts are expected (e.g., horizons that are hydraulically isolated from the works by very low-permeability strata).

It is also essential that the type of monitoring installation or device is appropriate to the conceptual model. In particular, for the case where the impact of concern is ground settlement resulting from pore water pressure reductions in low-permeability alluvial soils, conventional standpipe piezometers may not provide realistic monitoring results. Standpipe piezometers require a relatively large flow of water into or out of the soil in order to register a change in groundwater level. In low-permeability soils such as silts or clays, these instruments will respond only very slowly to changes in pore water pressures. Where it is necessary to accurately monitor pore water pressures in low-permeability compressible soils, consideration should be given to the use of "rapid response" instruments such as vibrating wire piezometers (VWP), which use electronic pressure transducers (see Section 6.6.3).

In the past, monitoring has been done manually by visiting each location. This has restricted the frequency at which data can be obtained economically. The availability of simple, cheap, and reliable datalogging systems (see Section 16.6) now provides the option of obtaining almost continuous records of parameters.

Figure 15.17 shows an example of the use of datalogging systems to monitor a private water supply borehole assessed to be potentially impacted by a nearby dewatering system. In normal operation, the groundwater level in the supply borehole fluctuates during the day, in response to the pump being turned on and off to meet the user's water demand. This means that, when manual readings of water level are taken on a daily basis (a commonly used frequency of manual readings), there is a lot of scatter in the water levels, depending on whether the pump was operating at the time of the reading. Such manual readings are shown in Figure 15.17a. This shows that, although a general reduction in groundwater level is indicated, the scatter in the data makes it difficult to estimate precisely the drawdown impact at the supply borehole. In contrast, Figure 15.17b shows the data for the same

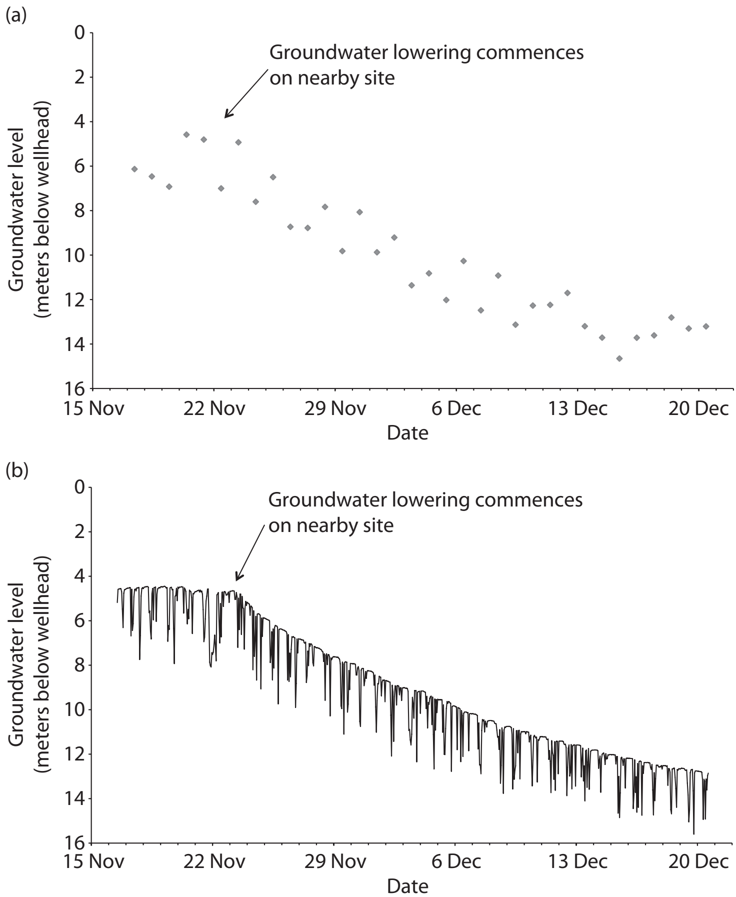

Figure 15.17 Monitoring of impacts on private water supply borehole. (a) Monitoring manual readings of water level taken once per day. The scatter in the data results from variations in water level in the borehole caused by the pump switching on and off in response to the user's water demand. (b) Monitoring using a datalogger equipment taking water level readings every hour. The more frequent readings show more clearly the variations in water level in the borehole caused by the pump switching on and off. These data allow the drawdown impact to be better identified.

period, but with readings taken at hourly intervals by a datalogger. This dataset clearly shows the drawdown impact on the supply borehole and allows much better estimation of the magnitude of the impact.

Where groundwater impacts are a potential concern, it is essential that adequate "background" or baseline data are obtained to provide reference levels against which any impacts can be measured. For major groundwater control projects, an extensive program of data collection often forms part of the design and planning stage of a scheme.

Streetly (1998) has pointed out the difficulties of establishing the true baseline conditions against which to assess impacts such as changes in groundwater level. Typically, even in the absence of dewatering activities, groundwater levels will vary in the short term (due to barometric changes, rainfall, or abstraction). In the longer term, groundwater levels may fluctuate due to variations in recharge and, ultimately perhaps, climate change. Without some indication of the magnitude and direction of these variations, interpretation of monitoring data is problematic. Possible solutions include the installation of "control" monitoring points, beyond the area anticipated to be influenced by the project. Alternatively, hydrographs of "distant" observation boreholes may be available from regulatory authorities (such as the Environment Agency in England and Wales); this is also a useful way of obtaining historical data for the time period before access to the site allows installation of dedicated monitoring points.

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Chapter 16

Monitoring and maintenance of groundwater lowering systems

16.1 INTRODUCTION

Any groundwater lowering system will need, to some degree, monitoring and maintenance measures to ensure effective operation. This chapter describes the methods commonly used for monitoring of groundwater lowering works, including the use of dataloggers and automatic control systems. Maintenance issues are also discussed, as are methods of sealing boreholes on completion. The problems of corrosion, encrustation, and biofouling, which sometimes result in the gradual deterioration of system performance, are described.

16.2 NEED FOR MONITORING

Once in operation, a groundwater lowering system is the result of a great deal of effort by a number of people. It is a complex system dependent on a diverse range of hydrogeological, hydraulic, chemical, mechanical, and human factors, but it will have a clear aim: to lower groundwater levels sufficiently to allow the construction works to proceed. It would be silly, having invested so much time and money in a system, not to monitor it to check that initially, and on a continuing basis, these aims are achieved. Yet, many systems are not adequately monitored, leading to poor performance and loss of time and money, not to mention a stressful time for those concerned.

Groundwater lowering systems should have specific targets—maximum allowable groundwater levels at particular locations. They achieve these targets by pumping groundwater. The two most important parameters to be monitored are water levels (see Section 16.3) and pumped flow rate or discharge (see Section 16.4). Other parameters may also need to be monitored (see Section 16.5).

Monitoring is important throughout the operational life of a system, but especially so soon after the start-up of pumping. There have been many cases where carefully designed systems have not initially achieved the target water levels. This potentially embarrassing eventuality may result from some very simple problems with the mechanics of the system or the way it is operated. Occasionally, problems arise if ground and groundwater conditions differ significantly from those indicated by the site investigation—so-called "unforeseen ground conditions." Monitoring during the early stages of pumping is vital to allow potential problems to be identified and solved.

In some cases, particularly on large projects or where site investigation information is limited, monitoring of system performance can be used as part of the observational method of design (see Section 7.3). The observational method involves developing an initial design based on the most probable conditions, plus contingency plans to allow modification of the system in light of conditions actually encountered. Adoption of the observational method can allow fine-tuning of the system in difficult ground conditions. There may be a temptation to install only the bare minimum pumping capacity or number of wells to achieve the drawdown. This temptation must be resisted, and allowance should be made for standby equipment (see Section 16.7) and long-term deterioration of the system (see Section 16.9).

Whatever monitoring is carried out, merely taking the readings is not enough. They need to be reviewed by personnel who can interpret them appropriately. Plotting of long-term trends of groundwater levels or discharge flow rates can aid the identification of potential problems or anomalies. On many projects, this need not be done by a groundwater lowering specialist as such. The main contractor's site engineer could review them, provided they have been briefed by the system designer or installer as to what factors are important and if there are any particular targets that must be achieved. Whoever reviews the data, specialist advice should be sought if there is any uncertainty about the effectiveness of system performance.

16.3 MONITORING OF WATER LEVELS

The most basic form of monitoring is the measurement of water levels in observation wells to determine whether groundwater levels have been lowered to the target levels. The water level is most commonly measured using a "dipmeter," "dipper," or "water level indicator" (Figure 16.1); the process is known as "taking dip readings." The dipmeter consists of a graduated cable or tape with a probe at its tip. The probe is lowered down the well until it touches the water surface; the water completes a low-voltage circuit, and a buzzer or light is activated on the dipmeter reel.

There is an art of obtaining reliable dip readings. It is best to just gently lower the probe to the water surface (when the signal will activate), then raise the probe a little (the signal should cease), and lower the probe back

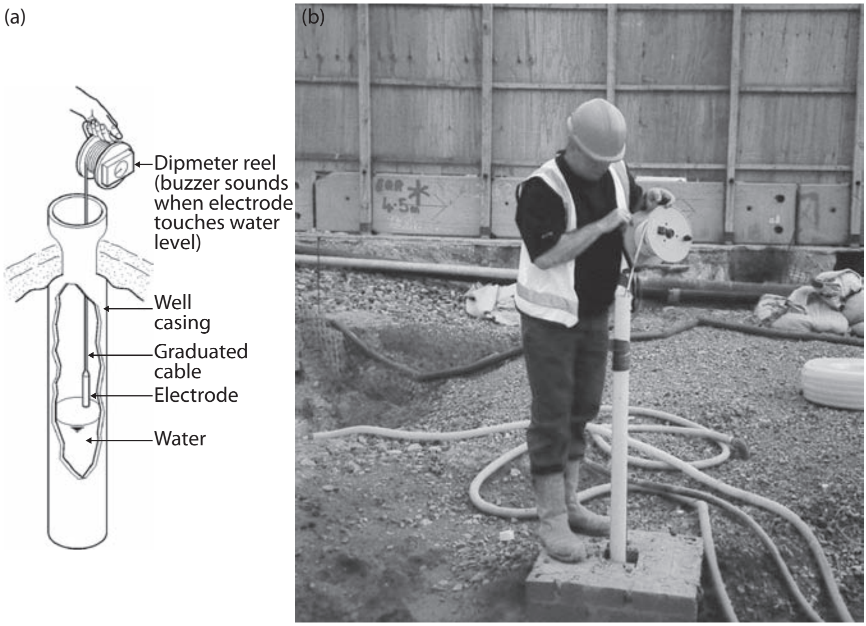

Figure 16.1 Dipmeter for measuring depth to water in a well or piezometer. (a) Schematic view of dipmeter in use. (From Preene, M. et al., Groundwater control—Design and practice, Construction Industry Research and Information Association, CIRIA Report C515, London, 2000. Reproduced by kind permission of CIRIA: www.ciria.org) (b) Dipmeter used to record water level in a monitoring well. (Courtesy of WJ Groundwater Limited, Bushey, U.K.)

into the water to confirm the reading. The graduated tape is used to determine the depth to water. Commercially available dipmeters can be used in bores of 19 mm or greater internal diameter.

The most useful records of groundwater levels are those taken from unpumped wells or observation wells, not from pumped wells. This is because a pumped well may show a significantly lower water level than in the surrounding aquifer due to the effect of well losses (see Section 3.5). Observation wells constructed as standpipe piezometers respond to water levels in one stratum only (see Section 6.6) and tend to give more representative groundwater levels than unpumped dewatering wells, which may exhibit a hybrid or average water level influenced by more than one stratum. If taking dip readings in a pumped well, it is best to install a plastic dip tube of 19–50 mm diameter down which the dipmeter probe is lowered. This avoids the tape getting stuck around the pump, riser pipe, or power cable.

Although the measurement of groundwater levels by dipping is a very simple process, it is possible that misleading readings may be generated. Possible causes for rogue readings include

1. *Clogged observation wells*. If the well has a clogged screen, the water level inside it will not be representative of the groundwater around it. Once a few days' readings are available, clogged wells are often easy to identify, because readings will tend to be constant with time and will not reflect changes in drawdown shown in other, unclogged, wells. Wells can sometimes be rehabilitated by flushing out with clean water or compressed air.
2. *Cascading of water into a well.* If a well has a large depth of screen and the water level inside the well is drawn down, water may cascade into the well from the upper parts of the screen above the true water level. When the dipmeter probe is lowered down the well, it may signal when it touches the cascading water, indicating a water level higher than the true level. Dipmeters with "shrouded" probes are less prone to this problem and should be used if at all possible.
3. *Saline water.* The dipmeter relies on the groundwater acting as an electrolyte to conduct the low-voltage current and trigger the signal. Saline or heavily mineralized waters are more conductive and can cause problems. The dipmeter may be triggered erroneously by small moisture droplets on the side of the well casing or may signal continuously once water gets onto the probe. Obtaining reliable readings in these conditions depends on the skill (and patience) of the operator. It helps to dry the probe on a cloth between readings and to repeat the reading at each well several times to be sure that a "true" reading has been obtained. Some dipmeters have an internal sensitivity adjustment. If this is turned to a minimum setting, the dipmeter may be more reliable in saline wells. Occasionally, poorly conducting water (with a low total dissolved solids [TDS]) may be encountered, where contact with the water does not complete the circuit and activate the dipmeter. This has been solved by the simple expedient of adding a packet of salt to the observation well.
4. *Errors in recording readings*. The person taking the readings may have made an error. The graduated tape of a well-worn dipmeter can be difficult to read when used in the field. It is not uncommon for the centimeter part of the reading to be correct, but with the meter part in error by 1 m. If the operator is recording all the readings on a sheet or notebook, this error may be repeated for subsequent readings. This is because the operator may expect each new reading to be similar to the last and will copy the meter portion of the reading from one record to the next. When checking the data, any sudden 1-m changes in level may indicate this type of error.

5. *Malfunctioning dipmeters*. Dipmeters sometimes malfunction. A dipmeter that does not work (perhaps because the battery is flat) is a practical problem but does not generate false readings. A more subtle problem is when the dipmeter tape is damaged, and the conductors are broken and exposed above the probe. The dipmeter will not signal when the probe touches water but may do so when the exposed conductor reaches the water, indicating a lower water level than is actually present.
6. *Modified dipmeters*. Dipmeters are sometimes repaired by shortening the graduated tape and rejointing the probe. If the operator is not aware of this, the recorded water levels will be deeper than the actual levels, with the difference being equivalent to the shortening of the tape. It is not a good practice to shorten dipmeter tapes; they should be replaced with a new tape of the correct length.

It is not always straightforward to spot rogue readings, but it becomes easier with experience. If in doubt, treat the readings with caution and investigate how they were taken. Much information can be gained by speaking directly to the person who took the readings. If the long-term trends of water levels are plotted and compared with any changes in pumping, rogue readings are often obvious.

In the field of hydrogeology for water supply, water levels are often measured at weekly or even monthly intervals (Brassington 2006), which is sufficient to identify long-term trends. For temporary works groundwater lowering applications, water level monitoring is normally carried out much more frequently. This is because short-term effects (such as fluctuation in drawdowns resulting from pump failures) are of greater interest. Typically, groundwater levels in all observation wells or piezometers should be recorded at least once per day, on every day of operation. On sites where the groundwater lowering system is critical to the stability of the works, groundwater levels may be recorded more frequently, perhaps two or three times per shift. If very frequent monitoring is necessary (perhaps on sites where water levels vary due to tidal effects), consideration should be given to the use of automatic datalogging equipment (see Section 16.6 and Figure 15.17).

16.4 MONITORING OF DISCHARGE FLOW RATE

The discharge flow rate pumped by the system is another vital measure of performance. The way the flow rate is measured depends on the pumping method in use:

1. Wellpoint systems—total flow rate generally measured at a common discharge point.

2. Deep well systems—flow rate may be measured at a common discharge point or measured for individual wells and then added together.
3. Ejector systems—flow rate may be measured at a common discharge point, or measured for individual wells (calculated as the outflow from an ejector minus the inflow) and then added together.

The discharge flow rate is commonly measured by one of the following three methods:

1. *Flowmeters*. Proprietary flowmeters may be of the totalizing type (which record total volumes of flow; flow rate is calculated from two readings at known time intervals) or the transient type (which measure flow rate directly). The most commonly used devices are propeller or turbine meters (where the flow rotates blades inside the meter) and magnetic flowmeters (where the water flows through a magnetic field, inducing a voltage proportional to the flow rate). All meters should be installed in the pipework in accordance with the manufacturer's recommendations. Flowmeters should generally be located away from valves and with adequate lengths of straight pipe on either side (typical requirements are a straight length of 10 pipe diameters upstream and five downstream). Flowmeters may require periodic maintenance or recalibration.
2. *Weir tanks*. This method is rugged and reliable in the field and involves passing the flow through a tank fitted with a V-notch (Figure 16.2) or rectangular weir and measuring the depth of water over the weir. Calibration charts allow this measurement to be converted to flow rate. V-notch weirs are the most common type in use; calibration charts for this type of weir are given in Appendix 5. The method is suitable for the routine estimation of flow rate for monitoring purposes, but it can be difficult to achieve high precision with weirs installed in small tanks (BS 3680:1981).
3. *Volumetric measurement*. This simple method estimates flow rate by recording the time taken to fill a container of known volume (the method is sometimes called the container or bucket method). It is important that a sufficiently large container is used to avoid too much water being lost by splashing and to ensure that the container fills slowly enough for accurate timing. For low flows (generally less than 5 L/s but perhaps up to 10 L/s), this technique gives reasonable accuracy using equipment that is cheap, rugged, and easily portable. Ideally, timing should be by stopwatch, and a relatively large container (40–200 L) should be used. The measurement should be repeated three times, and the average flow should be recorded.

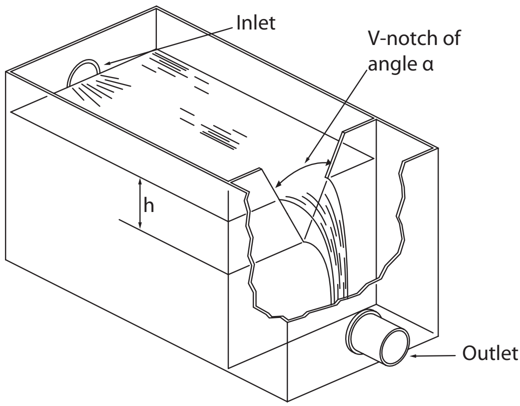

Figure 16.2 V-notch weir for the measurement of discharge flow rate. The depth of water *h* over the weir is measured above the base of the V-notch. The position of the measurement should be upstream from the weir plate by a distance of approximately 0.1–0.7 m but not near a baffle or in the corner of a tank. Baffles may be required to smooth out any surges in the flow. (From Preene, M. et al., Groundwater control—design and practice, Construction Industry Research and Information Association, *CIRIA Report C515*, London, 2000. Reproduced by kind permission of CIRIA: ww.ciria.org.)

If working in developing countries or in remote locations, the weir tank or volumetric methods are preferred for their simplicity. A jammed or broken flowmeter could be an embarrassment when located remote from the nearest service agent or replacement meter.

16.5 OTHER PARAMETERS THAT MAY BE MONITORED

In addition to groundwater levels and flow rate, more complex projects may require other parameters to be recorded. Table 16.1 is a comprehensive list of parameters that can be observed; it is unlikely that the full list would be monitored on a single project. Oftentimes, on sites where there are potential concerns about changes in groundwater quality caused by groundwater pumping, perhaps due to saline intrusion or migration of groundwater contamination (see Section 15.4.9), it is necessary to monitor certain chemical parameters of the pumped water directly in the field. Field monitoring techniques for groundwater quality are discussed in Section 3.9.2.

Table 16.1 Observable parameters for dewatering systems

Parameter	*Method*	*Comment*
Soil stratification	Well borehole log Observation of jetting water returns	Should always be checked against site investigation
Drawdown	Dipmeter monitoring or datalogging of observation wells (standpipe or standpipe piezometer) Dipmeter monitoring or datalogging of unpumped well	Generally essential; should be monitored daily
Flow rate, system total	V-notch weir Flowmeter Volumetric measurement	Generally important; should be monitored daily
Flow rate, individual well	V-notch weir Flowmeter Volumetric measurement	Important for well and ejector systems only; monitoring interval of 1–6 months
Water level in pumped well	Dipmeter monitoring of dip tube in well	Useful check on well performance
Mechanical performance	Vacuum (wellpoints) Supply pressure (ejectors) Discharge back pressure Engine speed (diesel pumps) Power supply alarms	Important for maintenance and/or monitoring continuous running, but data are required only if flow rate or drawdown is unsatisfactory
Water quality	On-site (e.g., pH or specific conductivity) Off-site (laboratory testing)	Necessary to assess clogging potential and environmental impact
Suspended solids	Condition of settlement tank or lagoon; turbidity or suspended solids content of discharge water	Always recommended to check for fines removal
Settlement	Preconstruction building condition survey Level monitoring of selected locations Crack monitoring of selected locations	Sometimes necessary if risk of damaging settlements is significant
Tidal effects	Regular monitoring (at 15–60 min intervals) of drawdown for a minimum period of 24 h	Provides a useful check on data; detailed analysis of significance is complex
Rainfall	Rain gauge on site Data from regional weather stations	Can be important for pumping test, or for assessing the impact of dewatering pumping on regional groundwater levels
Barometric pressure	Barometer Data from regional weather stations	Can be relevant for pumping test or for assessing the impact of dewatering pumping on regional groundwater levels

Source: Modified from Roberts, T.O.L., and Preene, M., *Géotechnique*, 44(4), 727–734, 1994.

16.6 DATALOGGING SYSTEMS

On most projects, particularly those of a smaller scale, the collection of monitoring data is generally carried out by a technician manually dipping water levels, reading flowmeters, and recording the results in notebooks or record sheets. This can be avoided by using electronic datalogging equipment, connected to appropriate sensing devices to record the required data. The datalogger stores the data until the operator collects (or downloads) the data for inspection and interpretation. As each year passes, available datalogging equipment is becoming cheaper, smaller, and more versatile. Equipment that, only a few years ago, might have been suitable for large-scale projects only is now cost-effective on even simple schemes.

The use of dataloggers can reduce the number of personnel needed for monitoring, especially if regular monitoring during day and night shifts is required. By downloading the data directly, there is no need for someone to laboriously key in manually recorded readings, and the time and effort required to go from data gathering to plotting and analysis of data can be minimized. If Internet connections (either by landline or by mobile phone) are available at the site, rapid dissemination of complete datasets is possible.

The most common application of datalogging equipment is to record groundwater levels. An electronic pressure transducer is installed below the water level in the well (Figure 16.3). Such devices typically operate on the vibrating wire principle (and are commonly known as vibrating-wire piezometer instruments or VWP), where the transducer contains a

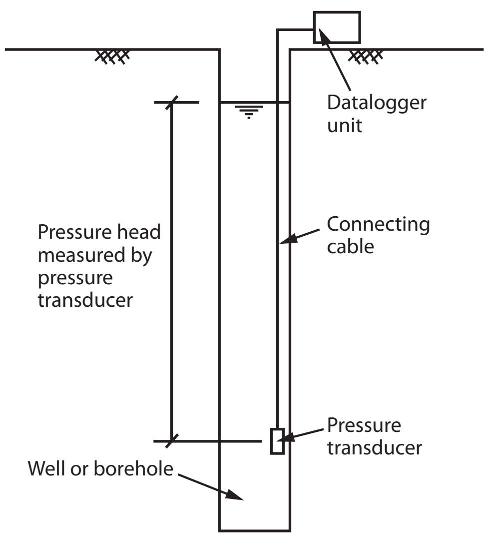

Figure 16.3 Schematic view of water-level datalogger.

diaphragm in hydraulic connection with the groundwater. Changes in water pressure deflect the diaphragm and an associated tensioned steel wire (the vibrating wire). The deflection changes the frequency response of the vibrating wire when excited electrically (via a connecting cable) by a readout unit or datalogger. Some of the most useful dataloggers are "standalone," that is, they have sufficient battery power to operate for extended periods without an external power supply. Dataloggers can be multiplex (serving several transducers) or single station (serving one transducer only).

The pressure transducer is lowered down to a significant depth below the lowest anticipated drawdown level; typically, for shallow installations, the transducer is suspended from its own cable. The transducer records the pressure above its own level; thus, an on-site calibration is required to allow pressure readings to be converted to water levels. This may be done by accurately recording the depth at which the pressure transducer is suspended in the well. Alternatively, if the water level is manually measured at the same time as the datalogger takes a reading, the effective transducer depth can be estimated by adding the datalogged pressure head (measured by the transducer) to the observed depth to water. The datalogger should be installed and calibrated in accordance with the manufacturer's guidelines; some types of pressure transducers are affected by barometric pressure changes, and appropriate corrections may need to be applied. Oftentimes, a "control" instrument is part of the system, sat in a small reservoir of water at ground level. This instrument will record the background fluctuations in observed water pressure due to barometric effects. The trace from the control instrument is then used to correct the data from the instruments in the wells to obtain the true groundwater pressure changes.

Flow rates can also be datalogged if the flow-measuring device produces a suitable electronic output that can be fed to the datalogger. This is normally possible with magnetic flowmeters and certain types of propeller or turbine flowmeters.

The use of datalogging systems has the potential to generate huge volumes of data, which can be disseminated very rapidly, via the Internet, to various parties. This is not necessarily a good thing. There is a real danger that "data overload" will occur and the use of dataloggers will actually be counterproductive and will make the identification of key factors and potential problems more difficult. It is essential that adequate procedures are put in place for the management and review of field data. Holmes et al. (2005) give a good example of data collation and management from a large project in the United Kingdom.

Datalogging systems can be useful for gathering data during site investigation pumping tests (see Section 6.7 and Appendix 3), as well as for operational monitoring.

16.7 MECHANICAL FACTORS AND AUTOMATION

The mechanical and electrical plant in the system will require periodic maintenance, depending on the type of equipment and the way it is being used. Diesel-powered pumps or generators will require regular fuelling and checking/replenishment of lubricant and coolant levels. Electrically powered equipment typically requires less maintenance but should be tested regularly (see BS 7671:2001).

The provision of standby pumps or power supply should be considered for all groundwater lowering systems where relatively short interruptions in pumping will cause instability or flooding of the excavation. Wellpoint and ejector systems, which have relatively few running pumps, often have standby pumps provided; one standby for every two duty pumps is typical. Deep well systems, which may have many pumps, often do not have standby pumps provided. This is acceptable, provided that the system design has allowed for a few extra wells over and above the minimum number required. This way, the failure of one or two submersible pumps will not cause a problem while the pumps are replaced. Where systems are electrically powered (from mains or generator), a standby generator should be provided as a backup.

Standby pumps and generators should be tested regularly (at least weekly) by running on load. If possible, it is a good practice to use the equipment in rotation, whereby a unit is run as the duty unit for 1 week, then, acts as a standby unit in the following week, and so on.

Electronic equipment is available to allow systems to be alarmed and, to some degree, automated. Sensors similar to those used in datalogging can be used to monitor groundwater levels, pump vacuum, ejector supply pressure, or the failure of the power supply or of individual pumps. The output from these sensors can be linked to the monitoring equipment, which can send an alarm signal (which could be via a siren, flashing beacon, radio, telephone, or Internet connection) to warn staff of problems and alert them to the need for remedial action. The remedial action might typically be to switch on the standby pumps or power supply and restart the system. Although this could be done manually by someone called out to the site, the next logical step is to automate this process and allow the system to activate the standby plant in response to the alarm signal. This is the basis of automatic mains failure (AMF) systems. Such systems are used on only a small minority of groundwater lowering systems but have been an established practice on some of the larger water supply wells since the 1980s. Electronics and computer technology are advancing constantly; it is probable that, in the near future, automation and remote monitoring will be applied to a much greater proportion of systems than at present.

16.8 BACKFILLING AND SEALING OF WELLS ON COMPLETION

Upon completion of the groundwater lowering works, after the pumping equipment has been removed, wells may need to be backfilled and capped off. This is required to

1. Remove the safety hazard of having open holes
2. Prevent the well acting as a conduit for surface contamination to reach the aquifer (this is especially important for wells penetrating aquifers used for public supply; see Section 15.5.1)
3. Prevent uncontrolled flow of groundwater between strata penetrated by the well

In some cases, particularly shallow wellpoint systems penetrating one aquifer only on uncontaminated sites, the wells may be left unsealed, with the wellpoint riser simply cutoff below surface reinstatement level. In contrast, almost all deep wells and many ejector wells will need to be sealed.

Sealing of wells first involves removing the pumps and any headworks from the wells. The well casing and screen are normally left in place but are cut off just below ground level. The well bore is then backfilled with appropriate material. The materials used to backfill a well must be clean, inert, and nonpolluting. Suitable materials include clean sands and gravels, bentonite, or cement grout (grouts should generally be placed via tremie pipes). It may be necessary to obtain agreement from the environmental regulatory authorities that the sealing methods proposed are adequate. In England and Wales, the Environment Agency (2000) has produced guidelines on best practice.

16.9 ENCRUSTATION, BIOFOULING, AND CORROSION

Wells pumping groundwater for extended periods may show a gradual reduction in performance (in the form of loss of yield or increase in drawdown in the well) during operation. The loss of performance may result from clogging due to encrustation (precipitation of chemical compounds) or biofouling (bacterial growth and associated processes). In addition, if the groundwater is saline or brackish, corrosion of metal components (such as pumps or pipework) may be of concern.

These effects have long been recognized in water supply wells, which have working lives of several decades. Such wells (and associated pumps and pipework) are typically rehabilitated at periodic intervals to ensure that performance does not reduce to unacceptable levels (Howsam et al. 1995). Gradual reduction in performance is less of a problem for temporary works groundwater lowering systems, largely because of the shorter pumping periods involved. Nevertheless, there have been a number of instances where systems operated for periods between several months and up to a few years have been affected. An understanding of the factors involved is useful when planning for the maintenance of systems, particularly for permanent or long-term systems (see Chapter 14).

The following sections mainly discuss clogging effects in wells, but problems can also occur with the encrustation or biofouling of pump internal components, pipework, and flowmeters.

16.9.1 Chemical encrustation

Groundwater contains, to varying degrees, chemical compounds in solution (see Section 3.9). These compounds may precipitate in and around the well, being deposited as insoluble compounds to form deposits of scale on well screens and pumps. The water experiences a drop in pressure and may be aerated by cascading as it enters a well; these are ideal conditions for precipitation.

The principal indicators of the encrustation potential of groundwater (from Wilkinson 1986) are

1. pH greater than 8.0
2. Total hardness greater than 330 mg/L
3. Total alkalinity greater than 300 mg/L
4. Iron content greater than 2 mg/L

Common deposits include calcium and magnesium carbonates, or iron or manganese oxides. For temporary works applications, chemical encrustation on its own is rarely severe enough to affect operation; problems normally occur when encrustation is enhanced by bacterial action, as described in subsequent sections.

When problems are first suspected, visual inspection, either of the equipment removed from the well or the well itself (via a closed-circuit television survey), can help identify the nature of the problem. The color of the encrustation deposits can give some indication of the type of deposit:

Black deposits	Iron sulfide or manganese deposits
Dark to reddish brown deposits	Ferric iron deposits
Light brown deposits	Mixture of calcium and manganese or iron carbonate
Very pale or white deposits	Calcium carbonate

Such visual inspections should be supported by chemical testing of the encrustation deposits and of the groundwater.

If groundwater analyses suggest that it may be a problem, the wells should be designed to have as low a screen entrance velocity (see Section 10.3) as possible. If encrustation becomes a problem once the system is in operation, acidization or chemical treatment of the wells, followed by air lifting or clearance pumping, may help loosen and remove deposits (Howsam et al. 1995).

16.9.2 Bacterial growth and biofouling

Groundwater naturally contains bacteria; problems occur because the wells and pipework forming a groundwater lowering system may offer an environment in which these bacteria can thrive. The most common and, hence, most problematic bacteria are the iron-related species such as *Gallionella* or *Crenothrix*. According to Howsam and Tyrrel (1990), these are sessile bacteria; this means that they attach themselves to surfaces. The action of the bacteria will cause a biofilm to develop on a surface. A biofilm consists not only of bacterial cells but also of proportionately large volumes of extracellular slime. This can trap particulate matter and detritus from the water flowing by and provides an environment for the precipitation of iron, manganese oxides, and oxyhydroxides. The most common form of biofilm (also known as biomass) is a thick red-brown gelatinous slime or paste that builds up in wells, pumps, and pipework.

The practical result of this is that, if conditions are favorable for bacterial growth, a system may become biofouled. The biofilm can be surprisingly tenacious and may clog a system if not removed or controlled in some way, dramatically reducing performance (Figure 16.4). As biofouling deposits build up, the discharge flow rate will decrease. If no action is taken, the drawn-down groundwater levels will rise and may result in instability or flooding of the excavation. Regular monitoring of flow rate and water levels is essential for the diagnosis of these problems. A program of periodic rehabilitation of the system may be necessary to ensure continued satisfactory operation.

Howsam and Tyrrel (1990) state that the following conditions may give an increased risk of biofouling: the use of iron or mild steel in the system (increasing iron availability); intermittent pumping, large well drawdowns, and cascading of water into the well (all increasing oxygenation); and high flow velocities, such as through well screen slots or at valves (increasing nutrient uptake). Unfortunately, many of these conditions are almost unavoidable in groundwater lowering systems. Therefore, biofouling should be considered as an operational risk for most systems.

Consider the conditions needed for the growth of iron-related bacteria (Howsam and Tyrrel 1990):

1. *Nutrients*. Bacteria, in general, need carbon, nitrogen, phosphorus, and sulfur. Many natural groundwaters contain sufficient levels of these nutrients to sustain significant biofilm growth.

Figure 16.4 Biofouling of submersible pump due to iron-related bacteria. The submersible pump and riser pipe shown have been removed from a dewatering well after several weeks of operation. The upper part of the pump is coated with a thick red-brown biofilm slime.

2. *The presence of an aerobic/anaerobic interface.* This is typically formed by a well, where anaerobic aquifer water can come into contact with oxygen. This interface is the point at which aerobic bacterial activity and chemical iron oxidation is initiated.
3. *The presence of iron in the groundwater.* The presence of dissolved iron is necessary, because the oxidation of the iron provides energy for the bacteria's metabolism. This action precipitates the insoluble iron compounds, giving the characteristic red-brown color to the biofilm.
4. *Water flow.* The flow of water transports nutrients to the biofilm. The faster the flow of water becomes, the more food is available for bacterial growth, and the faster the biofilm will grow. This is a key point, because it means that high-flow wells may biofoul more quickly than low-flow wells.

Conditions 1 and 2 will be present for most systems; thus, the likelihood of biofouling should be assessed from conditions 3 and 4—dissolved iron and rate of flow. Based on a number of case histories, Powrie et al. (1990) produced Table 16.2 in terms of those two factors.

Table 16.2 shows that the risk of biofouling problems is also dependent on the type of system. Wellpoint systems (where much of the pipework is under vacuum) do not provide a good environment for the growth of aerobic bacteria and are not especially prone to biofouling. Deep well systems using submersible pumps are prone to biomass building up on the outside of the pump and riser pipe, inside the pump chambers, and inside the riser pipe. Because the riser pipe is normally of relatively large diameter, clogging of the riser pipe is only a problem in severe cases; biofouling inside the pump is more of a problem, often leading to pump damage if not removed

Table 16.2 Tentative trigger levels for susceptibility to *Gallionella* biofouling

Pumping technique	*Susceptibility to biofouling*	*Concentration of iron in groundwater (mg/L)*	*Frequency of cleaning*
Wellpoints	Low	<10	Biofouling unlikely to present difficulties under normal operating conditions and times of less than 12 months
		>10	Biofouling may be a problem for long-term systems
Submersible pumps	Moderate	<5	6–12 months
		5–10	0.5–1 month
		>10	Weekly (system may not be viable)
Ejector wells (low flow rate; <10 L/min)	Moderate	<5	6–12 months
		5–10	Monthly
		10–15	Weekly (system may not be viable)
Ejector wells (high flow rate; >20 L/min)	High	<2	6–12 months
		2–5	Monthly
		5–10	Weekly (system may not be viable)
Recharge wells	Very high	Recharge wells are extremely prone to biofouling, which is likely to occur even if iron concentrations are below 0.5 mg/L To minimize biomass growth and encrustation, extreme care should be taken to avoid aerating the recharge water It is not uncommon for recharge wells to require cleaning on a weekly or monthly basis Recharge wells may not be viable at high iron concentrations	

Source: Modified after Powrie, W. et al., *Microbiology in Civil Engineering* (Howsam, P., ed.), Spon Press, London, 1990, pp. 341–352.

by cleaning. Ejector systems are particularly susceptible to clogging from biofouling; this is because the smaller diameter pipework that is generally used is easily blocked by biomass. Artificial recharge systems (see Section 11.9) are extremely vulnerable to biofouling.

If biofouling occurs and significant loss of performance has been identified by regular monitoring of groundwater levels and discharge flow rates, the system may need to be rehabilitated. Possible methods include

1. *Agitation, scrubbing, or flushing out of the wells*. Biomass may be removed from the wells by development techniques (see Section 10.7) that surge and agitate the well, thereby loosening the clogging material. Air lift surging and pumping, water jetting (using fresh water or a chlorine solution), and scrubbing (using a tight-fitting wire brush hauled up and down inside the well) have all been proven to be effective.
2. *Cleaning of pumps and pipework*. Submersible pumps, ejectors, and risers can be removed from wells and cleaned at ground level by jet washing or scrubbing. Pumps and ejectors may need to be disassembled to allow cleaning or replacement of internal components. Risers, headers, and discharge pipes may be cleaned by jetting or the use of pipe cleaning moles.
3. *Chemical treatment of wells and pipework*. In recent years, the use of chemical treatments to remove encrustation and to reduce the rate of subsequent regrowth has become widespread. Typically, this involves dosing the wells and the rest of the system (e.g., pipework) with specialist chemical solutions to kill the bacteria and then using methods 1 or 2 to remove the residue of biomass. Traditionally, many of the chemical treatments were chlorine based. However, well cleaning products (typically based on organic acids) are now on the market, which are highly effective yet are biodegradable and relatively environmentally friendly (Deed 2009). Some of these products are approved for use in drinking water wells. Chemical treatment is a specialist technique and should be carried out with care and due consideration for the handling and disposal of chemicals and effluents to reduce the health risks to the workforce and to minimize any environmental impacts.

Systems operating for long periods may need rehabilitation several times during their working lives. There is some evidence that regular cleaning on a "preventative" basis, that is, before the clogging and encrustation becomes very severe, is more effective than cleaning regimes that wait until the clogging reaches problematic levels before cleaning is carried out.

16.9.3 Corrosion

Metal components in the system (such as pumps, pipework, flowmeters, and steel well casings) may be subject to corrosion. Corrosion is a relatively minor problem in water supply wells but is rarely severe in the short term, as the abstracted groundwater is usually relatively fresh (i.e., low in chlorides). However, groundwater lowering systems may sometimes pump water that is significantly brackish or saline (see Section 3.9); this is likely in coastal areas or aquifers where saline intrusion has occurred. Systems pumping saline groundwater may be subject to very severe corrosion even during a few months of pumping. Figure 16.5 shows corrosion of submersible pumps

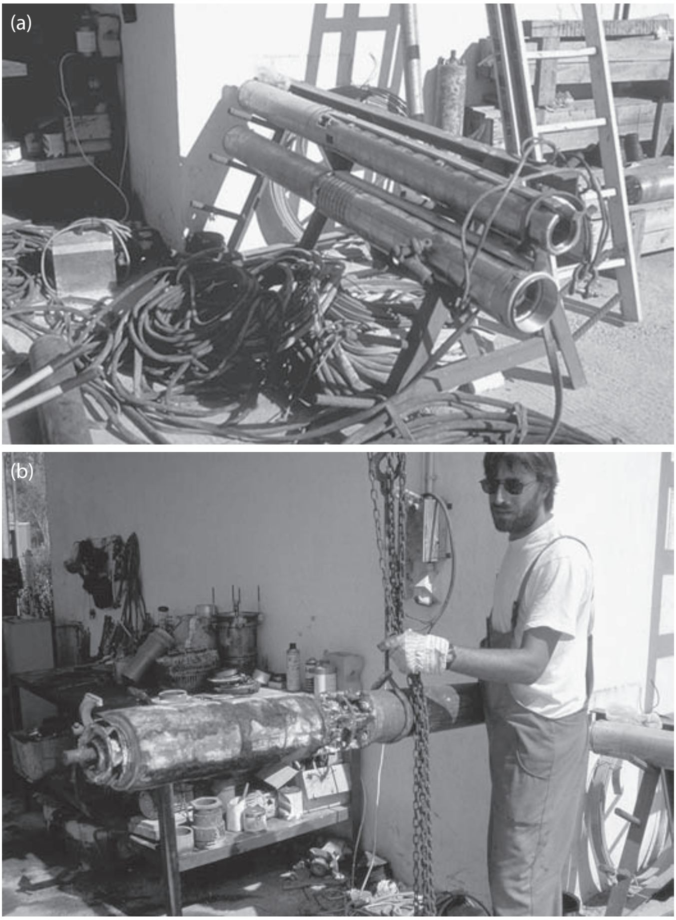

Figure 16.5 Extreme corrosion of submersible pumps in highly saline groundwater. (a) Pumps in pristine condition prior to use. (b) Heavily corroded pumps. Note: The previously shiny stainless steel pumps are now blackened and badly pitted.

(made from grade 304 stainless steel) after between 6 and 12 months of operation at a site with highly saline groundwater. This is an extreme example but does show that corrosion may occasionally be a problem.

The indicators of the corrosion potential of groundwater, according to Wilkinson (1986), are

1. pH less than 7.0
2. Dissolved oxygen (which will accelerate corrosion even in slightly alkaline waters)
3. The presence of hydrogen sulfide
4. Carbon dioxide exceeding 50 mg/L
5. TDS exceeding 1000 mg/L
6. Chlorides exceeding 300 mg/L
7. High temperatures

It should be recognized that these indicators are a guide only. Corrosion of groundwater abstraction systems is complex and may be influenced by other factors, including

1. *Electrochemical corrosion*. This is when corrosion is facilitated by the flow of an electric current. Two conditions are necessary: (1) water containing enough dissolved solids to act as a conducting fluid (electrolyte) and (2) a difference in electrical potential on metal surfaces. Differences in potential can occur on the same metal surface in heat- or stress-affected areas where the metal has been worked (such as threads or bolt holes) or at breaks in surface coatings such as paint. Such conditions allow both a cathode and anode to develop; metal is removed from the anode. Bimetallic corrosion occurs when two different metals are in contact and immersed in an electrolyte. This can affect pumps made from different metals; the more susceptible metal will corrode preferentially and may be severely affected.
2. *Microbially induced corrosion*. The growth of a biofilm (formed by iron-related aerobic bacteria) on the surface of metal pumps and pipework can allow an anaerobic environment to form below the film. If sulfates are present in the groundwater, the anaerobic condition can allow the growth of sulfate-reducing bacteria. These produce sulfides that are very corrosive to cast iron and stainless steel. Even though stainless steels are normally very resistant to corrosion, they become susceptible to attack by chloride ions under such conditions. The passive oxide layer, which normally prevents corrosion, cannot re-form in the anaerobic zone beneath the biofilm.

If corrosive conditions are anticipated, the materials used in the system should be chosen with care. Mild steel (as is commonly used for pipework) is very susceptible to corrosion, stainless steel (from which many submersible pumps are made) is less so, and plastics are inert to corrosion. To reduce corrosion risk, as much pipework (aboveground discharge pipes and belowground riser pipes) should be made from plastic. Consideration should be given to using pumps made from the more resistant grade 316 stainless steel rather than the more common grade 304. If microbially induced corrosion is suspected, regular cleaning of the system to remove the biofilm will remove the environment for this form of corrosion.

16.10 FAULT FINDING AND PROBLEM SOLVING

Similar to any complex system, there may be times when a groundwater lowering scheme does not adequately achieve its desired aims when switched on or if it does work effectively at first, its performance may deteriorate with time. Identifying system problems and figuring out how to put them right is a vital skill for any practical groundwater engineer. Most problems with groundwater lowering systems will probably have a mechanical, hydrogeological, or geotechnical cause, but it is worth thinking about all the factors that can have an effect:

1. *Ground*. The nature of the ground will clearly affect the way a groundwater lowering system performs.
2. *Pump*. An inappropriate or poorly performing pump may cause problems.
3. *Power supply*. Every pump needs a power supply, generally diesel or electric. Unreliable or underpowered units may be the cause of difficulties.
4. *Pipework*. If the water cannot get to or from the pump, a system will perform poorly.
5. *Environment*. External factors, such as changes in groundwater levels and bad weather, may have an influence.
6. *Human factors*. Systems do not generally run themselves; the way they are operated will affect their performance.

As with any daunting problem, the secret is to identify the possible causes and eliminate the irrelevant ones until only those directly linked to one's woes are left. It requires a logical approach and might even be enjoyable if it was not invariably carried out under pressure to solve the problem as soon as possible and allow the excavation to proceed.

Typical operational problems requiring correction include

1. During initial period of running
   a. *High-flow problem*. The flow rate is greater than the pump capacity and the target drawdown is not achieved.

   b. *Low-flow problem*. The flow rate is less than the pump capacity and the target drawdown is not achieved.
   c. *Lack of dry conditions in excavation*. When monitoring of groundwater levels indicates that the target drawdown has been achieved; but the excavation is still visibly 'wet'.
2. After extended running
   a. *Sudden loss of performance*. The system operates satisfactorily, but after some time, the flow rate or drawdown changes suddenly.
   b. *Gradual loss of performance*. The system operates satisfactorily at first, but performance deteriorates gradually with time.

To diagnose any significant problems, the raw material is monitoring data of the sort described in this chapter. If a system has not been adequately monitored, the first stage of any problem-solving process will involve gathering data about the system performance. This will take time, probably taxing the patience of all concerned; neglecting regular monitoring of groundwater lowering systems is a false economy in terms of both time and money.

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Chapter 17

Safety, contracts, and environmental regulation

17.1 INTRODUCTION

Groundwater lowering is the means to an end—the excavation and construction of belowground works in stable and workable conditions. It will be one of many diverse activities carried out on a construction site during the life of a project. As such, dewatering must

1. Be carried out safely and in such a way so as to minimize (as far as reasonably practicable) the risk of harm to the workers or others who might be affected by the works.
2. Have an appropriate contractual arrangement so that adequate information is available to allow design and construction and to ensure that all parties are aware of their rights and responsibilities.
3. Be executed with a view to minimizing or controlling any adverse environmental impacts and conforming to any specific legal restrictions imposed by environmental regulators.

This chapter presents brief details of the health and safety legislation in force in the United Kingdom and outlines key issues relevant to dewatering works. Typical contractual arrangements used to procure and manage groundwater lowering works are outlined, including the traditional contractor–subcontractor relationship, as well as alternative forms of contract. Finally, the environmental law in relation to dewatering is discussed, and the legal implications of dewatering works are outlined.

17.2 HEALTH AND SAFETY

In the United Kingdom, it is a legal requirement that all company and site management personnel ensure that the construction operations carried out by their company conform to safe working conditions and noise abatement regulations.

Since the beginning of the twentieth century, there has been concern over accidents and working conditions on building works. In 1904, a public inquiry was held, but it was not until 1926 that the first building regulations were passed into law. From time to time since then, building and construction regulations have been augmented and strengthened by various specific laws. Currently, the major legislation covering occupational health and safety of construction works include

1. Health and Safety at Work Act, etc., 1974
2. Management of Health and Safety at Work Regulations 1992
3. Construction (Design and Management) Regulations 2007

These pieces of legislation, which cover the whole construction process including design, are supported by more detailed legislation covering specific types of activities or risks.

Legislation will change with time, as technology and working practices evolve. This chapter provides only a brief overview of some significant issues. It is essential that the reader checks to see what legislation is current, consults the original regulations, and obtains specialist advice. This legislation applies to England and Wales, with Scotland and Northern Ireland having similar legislation. Naturally, this legislation is not applicable in other countries, but the legal duties of the various parties defined in the *Health and Safety at Work Act etc. 1974* are reflected in the legal systems of many other countries.

This section will deal specifically with health and safety issues particular to groundwater control works. Generic guidance on construction health and safety can be found in various publications (for example, see Health and Safety Executive 2006).

The remainder of this section on health and safety describes issues associated with

1. *Construction (Design and Management) Regulations 2007* (CDM regulations), which set out a philosophy of eliminating or reducing risk throughout the construction process
2. Working with hazardous substances
3. Use of electricity on-site
4. Lifting operations
5. Working at height
6. Work equipment and drilling rigs
7. Safety of excavations
8. Noise
9. Working with wells and boreholes

17.2.1 CDM regulations

The CDM regulations define the legal duties of the various parties to the construction process, including clients, their agents, designers, and contractors. In simple terms, the regulations require health and safety to be considered at every stage of a project, from design to construction, maintenance, and demolition. This is a change in focus from the traditional approach to health and safety, which concentrated on the construction phase and the role of the contractor.

The CDM approach requires designers to consider whether it is reasonably practicable to exclude or reduce potential hazards. The risk to health and safety arising from a particular aspect of the design has to be weighed against the cost of excluding that feature by

1. Designing to eliminate or reduce foreseeable risks to health and safety
2. Tackling the causes of risk at the source
3. Reducing and controlling risks by means that will protect everyone (rather than just the individual) and, therefore, yield the greatest benefit

The cost is based not only on financial considerations but also on fitness for purpose, environmental impact, and "buildability." The purpose of the regulations is not only to reduce possible hazards and risks but also to identify and record those that are unavoidable as a warning to subsequent parties in the design, construction, maintenance, and demolition process.

Detailed guidance on the application of the CDM regulations is given in the *Approved Code of Practice* by the Health and Safety Executive (2007) and in CIRIA Report C662 (Ove Arup & Partners 2007). The application of CDM to the design of groundwater lowering systems is discussed in the work of Preene et al. (2000).

In essence, the CDM regulations formalize what a good dewatering designer should be doing as a matter of course. First, considering the full range of options for a design and then, as far as practicable, designing systems that will be effective but can be installed without unnecessary risks to the workers or others—in other words, to consider the "buildability" of a system.

When considering options, the designer must look at details; for example, if a line of wellpoints requires the jetting crew to work next to an open deep cofferdam, could the line of wellpoints not be relocated to avoid the risk? However, the designer must also look at the bigger picture. If a sheet pile cofferdam is dewatered but relies on continued dewatering for stability, is that too risky an option? What if the power supply fails? What if the standby generator fails too? Such risks will need to be assessed against the factors discussed earlier. In this case, the options might be to avoid dewatering altogether and use a system based on groundwater exclusion. A multiple redundant pumping and power supply system may increase the cost but may produce a significant reduction in the risk of catastrophic failure of the excavation. These are the kinds of questions that should be part of the design process.

17.2.2 Working with hazardous substances

The *Control of Substances Hazardous to Health Regulations 2004* lay down the requirements for the assessment of risks from hazardous substances and for the protection of people exposed to them.

The regulations cover almost all potentially harmful substances (although some substances such as lead and asbestos are covered in a separate legislation). Suppliers of any chemical or hazardous substance must provide a data sheet to the purchaser, which must contain sufficient information to allow an assessment to be made of the risk to health from the use of the substance. Based on these assessments, employers have to ensure that, as far as reasonably practicable, the exposure of employees to hazardous substances is prevented or, if this is not possible, controlled. This may involve the use of personal protective equipment (gloves, eye protection, and respirators) appropriate to the hazard.

17.2.3 Use of electricity on-site

The use of electrical equipment on-site is controlled by

1. Electricity at Work Regulations 1989
2. Electrical Equipment (Safety) Regulations 1994

Portable equipment used on construction sites is generally limited to 110-V supply, arranged so that no part of the installation is at more than 55 V (single phase) or 65 V (three phase) to earth. These voltage limits are intended to reduce the severity of injuries if electrocution occurs. Many dewatering systems are electrically powered (particularly deep well and ejector systems) and use 415-V three-phase systems, in which each phase is at 240 V to earth. In the event of electrocution, injuries will be much more severe than with conventional equipment; unfortunately, fatal incidents do still occur in the construction industry.

Due to the high voltages involved, most dewatering electrical systems should be treated as permanent installations and should comply with the *Institution of Electrical Engineers Wiring Regulations* (which have been issued as BS 7671:2001). Electrical switchgear must be installed, commissioned, and regularly tested by an "authorized person" defined under the regulations.

Many injuries caused by electricity arise from equipment being worked on by inexperienced or authorized personnel or by poor communication between those working on the system. This can lead to personnel working inside panels or switchgear that they believe to be safe but are, in fact, live. A significant number of accidents occur this way. It is imperative that this situation is avoided by good communication, training, and management.

17.2.4 Lifting operations

Lifting operations, in one form or another, are an integral part of most dewatering or groundwater control projects. Some forms of lifting operation may be very obvious, such as the use of a crane to lower pumps or plant down into an excavation or where an excavator is used to lift a placing tube during wellpoint jetting. However, there are many other operations involving lifting, where sometimes insufficient attention is given to safe equipment and methods of working. Examples of less high-profile lifting operations include the use of lorry loaders to unload equipment from flatbed trucks or the use of hoists on drilling rigs to lift casings or drill tools. It is essential to realize that all lifting operations are potentially hazardous and must be carried out safely.

Lifting operations are governed by the *Lifting Operations and Lifting Equipment Regulations 1998* (known as the LOLER regulations). The regulations cover all lifting equipment that might be used on-site for lifting or lowering loads, including attachments used for anchoring, fixing, or supporting the equipment. Lifting equipment may include cranes, excavators, drilling rigs, forklift trucks and telehandlers, hoists, and mobile elevating work platforms, as well as lifting accessories such as chains, slings, and eyebolts.

The LOLER regulations require that

1. Lifting operations are carried out safely and are planned, organized, and performed by competent people.
2. The lifting equipment used is strong and stable enough for the particular use and is marked to indicate safe working loads.
3. The lifting equipment is positioned and installed to minimize any risks.
4. The lifting equipment is subject to ongoing thorough examination and, where appropriate, inspection by competent people.

Further information is given in the Health and Safety Executive (1998).

17.2.5 Working at height

Falls from height are one of the most common causes of serious injuries on construction sites. Working at height is defined as work in any place where a person could be injured by falling from it. In relation to dewatering activities, working at height can include working next to open excavations or activities where operatives need to adjust or repair pieces of equipment above ground level, such as on the mast of a drilling rig or on top of a groundwater treatment plant.

These activities are governed by the *Work at Height Regulations 2005*. The regulations establish the hierarchy for safe working at height as

1. Avoid work at height where practicable.
2. Use work equipment or other measures to prevent falls where working at height cannot be avoided.
3. Where the risk of a fall cannot be eliminated, use work equipment or other measures to minimize the distance and consequences of a fall should one occur.

Specialist equipment, which may be relevant to working at height, includes means of access for work at height, collective fall prevention (e.g., guardrails and working platforms), collective fall arrest (e.g., nets and air bags), and personal fall protection (e.g., work restraints, fall arrest, and rope access).

17.2.6 Work equipment and drilling rigs

Dewatering activities and the associated construction projects involve a wide range of work equipment, from large piling and drilling rigs, trucks, and plant down to hand tools, ladders, and knives. All these types of equipment present potential health and safety hazards and must be used safely.

Work equipment is subject to the *Provision and Use of Work Equipment Regulations 1998* (known as the PUWER regulations). The regulations require that an equipment is

1. Suitable for the intended use
2. Safe for use, maintained in a safe condition, and, in certain circumstances, inspected to ensure that this remains the case
3. Used only by people who have received adequate information, instruction, and training
4. Accompanied by suitable safety measures, for example, protective devices and markings

Although the PUWER regulations apply to a wide range of equipment used to install and operate a dewatering system, there are particular issues associated with drilling rigs. Drilling rigs use a lot of kinetic energy, in the form or rotating or falling drill tools, to break up soil or rock to form boreholes. If the human body interacts in the wrong way with the moving parts of a drilling rig, then serious injury or death can result. There have been cases where drilling crew members have lost fingers as a result of crush injuries caused by heavy drill tools or where operators have been killed or seriously injured by being entangled in the rotating parts of drilling rigs.

Since the late 1990s, there has been a focus, in line with the risk avoidance and risk reduction ethos of the CDM regulations (see Section 17.2.1), on modifying the design of drilling equipment to improve safety. Most commonly, this involves the provision of mesh guards or other protective devices around rotating or other dangerous parts of rigs. The guards must fully enclose the rotating parts in any areas where they could come into contact with personnel and be interlocked with the rig controls to prevent operation of the equipment while the guards are open to allow, for example, the addition of further rods to the drill string.

Further information is given in the Health and Safety Executive (2008) in relation to the PUWER regulations and by the British Drilling Association (2000) in relation to safety issues associated with drilling equipment.

17.2.7 Safety of excavations

A significant number of accidents occur in excavation works, particularly as a result of falls or resulting from collapses of poorly supported excavations. Legal requirements for safety in excavations are contained in the *CDM Regulations 2007*. A key requirement is that a competent person is required to inspect the excavation at the start of each shift while persons are working in the excavation. Additional inspections are required after any event that may have affected the strength and stability of the excavation. Further details of the requirements for safe working are given in the Health and Safety Executive (1999).

Practical guidance on methods of work in trenches up to 6-m depth is given in the work of Irvine and Smith (2001).

17.2.8 Noise

Pumps and dewatering plant are normally operational continuously day and night, 7 days a week. Any noise from the plant will be a potential health hazard to those working on the site and an annoyance to the public around the site.

The exposure of employees to noise is governed by the *Noise at Work Regulations 2005*, which sets various action levels in relation to daily exposure to peak noise levels; noise levels need to be assessed in the working area. Noise should be reduced at the source, but if this is not reasonably practicable, then appropriate hearing protection must be provided and worn.

The effect of noise on the public outside the site is covered by the *Control of Pollution Act 1974*, which gives local authorities statutory powers to set noise limits and allowable working hours.

Selection of dewatering plant has the greatest impact on ambient noise levels:

1. The quietest systems are those electrically driven from mains power. Deep well systems thus powered will be almost silent (because the pumps are belowground), and wellpoint or ejector systems will only generate a low "hum" from the rotation of the pump and motor.
2. If mains power is not available, an electrical plant should be run from supersilenced diesel-powered generators; modern equipment is very quiet.
3. For wellpoint systems, diesel-powered pumps are sometimes the only practicable option. Pumps with silenced or "hushed" prime movers should be used whenever possible not only to reduce ambient noise but also to reduce exposure to operatives. If pumps with unsilenced prime movers are used, noise levels should be reduced by reducing the engine revs as far as possible.
4. Acoustic screens can also be constructed around pumps, but these can cause problems with ventilation and overheating of pumps that have resulted, in extreme cases, in pumps catching fire. It is far better to start with suitably silenced equipment in the first place.
5. Noise from all items of the plant can often be reduced if regular maintenance and servicing of plant is undertaken to try and ensure that it is running smoothly and efficiently.

17.2.9 Working with wells and boreholes

Many groundwater lowering works will involve the construction, commissioning, maintenance, and, ultimately, decommissioning of wells. The hazards associated with wells and boreholes can broadly be subdivided into three types:

1. *Hazards during drilling, relating to the drilling and construction works themselves*. These include crushing, penetration, or impact injuries caused by equipment breakages or inappropriate handling of casing and health issues arising from ingestion or dermal contact with soil or groundwater on potentially contaminated sites.
2. *Hazards during operation*. During operation, pumps will be installed in most wells, reducing the risk of falls into all but the largest diameter wells. However, if any relatively large diameter wells are left without pumps during part of the construction period, they should be temporarily covered or capped off.

3. *Hazards following decommissioning*. These only arise if the wells are not adequately decommissioned by backfilling or capping off at the end of the project, as outlined in Section 16.8. Larger diameter wells, if left uncovered, clearly present a hazard. Even the smaller diameter wells (typically of a few hundred millimeters open bore) common on dewatering projects may present a hazard to small children if they can gain access to the site. It is essential that wells are decommissioned responsibly.

Guidance on safety during drilling works is given in publications by the British Drilling Association (2002, 2008). Particular guidance on working with large diameter wells is given in *Safety in Wells and Boreholes* (Institution of Civil Engineers 1972).

17.3 CONTRACTS FOR GROUNDWATER CONTROL WORKS

Groundwater lowering is a specialist process that, when measured in cost or manpower terms, is often only a tiny part of the overall construction project. Despite this, groundwater (and its inadequate or inappropriate control) has historically been the cause of many construction disputes. Suitable contractual arrangements are an important part of managing dewatering works for positive results. This section discusses the background to contractual issues and outlines some arrangements used in practice.

17.3.1 Need for contracts

As described throughout this book, groundwater control is highly dependent on the ground conditions, over which the designer has no control. It follows that groundwater control must run a greater risk of poor performance than, for example, reinforced concrete construction (where the designer can specify and control the materials used). Dewatering designers must gain their design information from the results of the site investigation (see Chapter 6). Sadly, it is still the case that not all investigations attain the standards that designers would aspire to; see *Inadequate Site Investigation* (Institution of Civil Engineers 1991) and *Without Investigation Ground Is a Hazard* (Site Investigation Steering Group 1993).

Because groundwater control works are often carried out at the very start of a construction project (e.g., for construction of foundations), any problems or delays at that stage can have serious knock-on effects for the rest of the project. A review of cost overruns on groundwater lowering projects is given in the work of Roberts and Deed (1994).

The specialized and perceived "risky" nature of groundwater lowering works has led many clients and main contractors to view dewatering as a "black art" best left to the cognoscenti. Apart from the very largest projects, many contractors prefer to subcontract dewatering works to specialist organizations that provide the expertise, experience, and equipment to carry out such work. It is important that an appropriate contract exists among the various parties and that the rights and responsibilities of each are clearly identified.

17.3.2 Traditional contract arrangements

In the United Kingdom, the traditional form of construction contract involved the client appointing a client's representative (called the Engineer under some forms of contract) to administer and supervise the works. Traditionally, the client's representative would also design the permanent works but not the temporary works, which were the remit of the contractor. The client's representative would, via a bidding or negotiation process of some sort, arrange for a contractor to undertake the works. The contractor would be employed directly by the client under a form of contract such as one of the editions of the Institution of Civil Engineers conditions of contract (known as the ICE conditions).

Groundwater lowering works are almost always classed as temporary works and are the main contractor's responsibility. These works are commonly subcontracted, and the contractor would employ a dewatering subcontractor. The subcontract between the dewatering company and the main contractor would typically be "back to back," meaning that the rights and responsibilities of each party apply to the other. In essence, this means that the dewatering subcontractor is effectively acting on behalf of the contractor and takes over their responsibilities relevant to the dewatering. It also means that, in the event of any changes or problems, the dewatering subcontractor has the same rights as the contractor to apply for additional time or money via the clauses in the contract.

One of the most common type of serious disputes in traditional dewatering contracts occurs when the groundwater lowering system does not achieve the target drawdown, and it is believed that the ground conditions may not be as represented in the site investigation data provided at the tender stage. The ICE conditions contain Clause 12 (commonly known as the "unforeseen ground conditions" clause), which allows the subcontractor and contractor to apply to the client's representative for additional time or payment. The "claims" process is often protracted, whereby the contractors have to demonstrate that the physical conditions or artificial obstructions encountered could not reasonably have been foreseen by an experienced contractor. This process can sometimes distract from the real problem of trying to finish the project and to deal with the conditions actually present in the ground. Some claims cannot be quickly resolved and may go to court and are finally resolved (one way or the other) several years after the end of construction.

17.3.3 Alternative forms of contract

From the 1990s onward, in the United Kingdom, there were moves away from traditional contracts, which were viewed as being too adversarial and leading to too many costly and time-consuming "claims." When construction problems occur, prompt and open sharing of information can be vital in developing solutions; this did not always happen under traditional contracts. Sometimes in the past, such contracts were applied in such a way that rather more effort was spent trying to apportion blame than on solving the problem.

At that time, reports by Latham (1994) and Egan (DETR 1998) reviewed the then performance of the U.K. construction industry and recommended specific improvements to planning and execution and promoted increased efficiency and integration among the different parties involved in projects. In hindsight, the concept of having more integrated planning on projects has often improved the way that groundwater control has been carried out.

Under the old, often adversarial, contractual system, the need to control groundwater was often left as a last-minute temporary works fix for the contractor (after many other aspects of the project had been finalized) and was procured as a cost-driven "distress purchase." If the integrated approach of Latham and Egan is followed, it is more likely that key constraints, such as the need to control groundwater, will be identified as risks early on in planning. This can allow rational assessment and open discussion among the various parties to construction of the potential risks and the way they could be managed. This opens up a wide range of options to control groundwater including, for example, redesign of the permanent works to reduce (or avoid completely) the need for groundwater control. The Channel Tunnel Rail Link, constructed in the United Kingdom from the mid-1990s onward, is a good example of how geotechnical engineering requirements, including the need to control groundwater, were some of the key factors considered throughout the design process when assessing options for structures below ground level (O'Riordan 2003).

In recent years, the nature of contracts has also changed as a result of increased use of so-called "design and build" contracts. These involve the contractor designing the permanent works as well as temporary works and change the nature of the relationships among the client, the client's representative, and the contractor.

Various nonadversarial forms of contract have been developed. A number of different schemes are possible, including

1. "Partnering"—which implies the development of longer-term relationships among the various parties including client, permanent works designer, contractor, and specialist subcontractors.
2. "Open book" contracts—where information is shared and all parties are kept informed of what is going on and are able to have input into relevant decisions.

These forms of contract can allow the "risks" of unforeseen ground conditions (or other factors) to be shared among the various parties in an open and transparent way. Dispute resolution procedures exist within such contracts to allow problems to be quickly highlighted and examined without the need for claims or other confrontational procedures.

When these forms of contract are used, the aim should be to control risks to the project. Any subcontractors should be selected on the basis not merely of cost but also in terms of quality, health and safety, environmental management, and the ability to meet the program timescale. It is also important that, by involving all the parties, expertise and experience can be pooled to solve problems as quickly as possible. The need to overcome problems in a timely manner cannot be overemphasized; on modern construction projects, many cost overruns result mainly from time delays rather than changes in methods. The control of risks in construction is discussed by Godfrey (1996).

17.3.4 Dewatering costs

Because of the varied nature of groundwater lowering works and the wide variety of ground and groundwater conditions, the development of generic costs is not easy. It is not possible to estimate dewatering costs, even on an approximate basis, from the quantity of water pumped, the volume of soil dewatered, or the depth of drawdown. Dewatering costs are more commonly broken down on an "activity schedule" basis. Some of the activities will be costed on a unit basis (e.g., per well, per meter of header pipe), whereas the costs during the pumping period—pump hire, fuel, supervision, etc.—will be time-related charges (e.g., per day or per week).

17.4 ENVIRONMENTAL REGULATION OF GROUNDWATER CONTROL

As described in Section 15.2, in relation to construction works, groundwater is viewed primarily as a problem, hence the need for groundwater control. In other contexts, groundwater is a resource used for public and private drinking water and for industrial use. In many countries, environmental regulations or laws exist to help safeguard groundwater resources.

This section describes the environmental regulatory regime applicable to England and Wales and the implications for the planning of groundwater control works.

The person or organization responsible for the groundwater control has certain legal obligations to ensure that consents and permissions are obtained from the regulators. Under the normal forms of contract, the party responsible is either the contractor or the client. The dewatering subcontractor or pump hirer is not normally responsible for the consents, but they should satisfy themselves, before work commences, that the necessary consents and permissions have been obtained.

There are two main facets to the legal requirements. The first deals with pumping of groundwater (termed *abstraction*) and is intended to make sure that the regulators can control groundwater abstraction to ensure that groundwater lowering systems do not cause nearby groundwater users to lose their supplies. The second deals with disposal of groundwater (termed *discharge*) and is intended to ensure that the pumped water does not itself cause pollution.

17.4.1 Groundwater protection

Groundwater protection is the collective term for the policy and practice of safeguarding the groundwater environment. This covers protection of both groundwater resources—the quantity of groundwater available for use (to ensure that the aquifers do not "dry up")—and of groundwater quality (to ensure that groundwater is not contaminated by man's activities).

The regulatory regime for groundwater protection in England and Wales is described in *Groundwater Protection Policy and Practice* (Environment Agency 2008).

One way of protecting groundwater resources is to require abstractors to be licensed. The licensing process allows the regulators (typically a governmental or quasi-governmental body) to scrutinize the applications and set limits on abstraction rates or, perhaps, even prohibit abstraction in certain areas. Licensing of abstractions is a relatively modern concept, dating from the second half of the twentieth century. The history of the chalk aquifer beneath London is described in Section 3.6, where unregulated abstraction in the late nineteenth and early twentieth centuries resulted in large and widespread lowering of the piezometric level, followed by a gradual rise in groundwater levels when abstraction rates were reduced (Figure 17.1). Such a situation would have been unlikely to occur had a system of regulation been in place at that time; modern regulators are, of course, left with the legacy of managing rising groundwater levels beneath London.

Overabstraction of groundwater resources has occurred in several other British cities and in other countries. In Thailand, for example, literally thousands of wells were drilled beneath Bangkok between the 1950s and

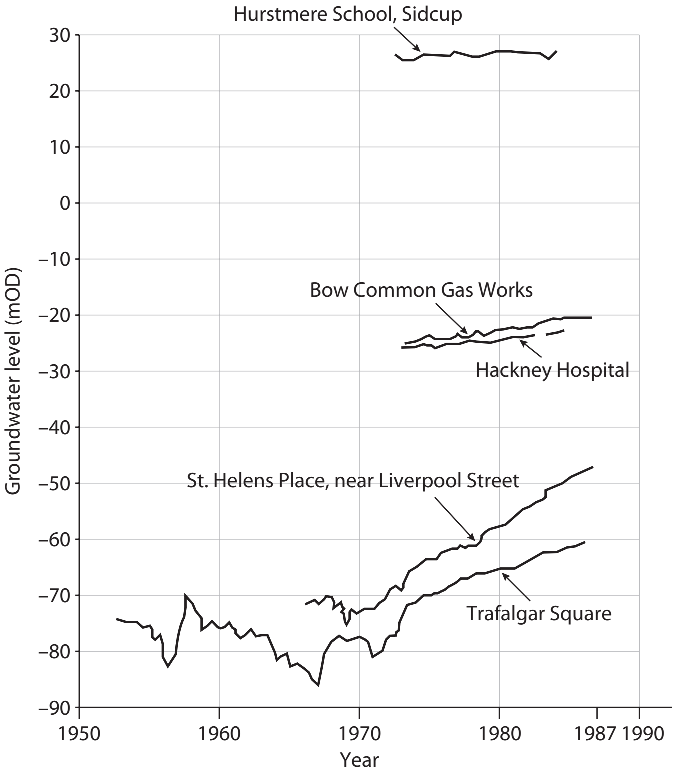

Figure 17.1 Rising groundwater levels beneath central London. (Redrawn from Simpson, B. et al., The engineering implications of rising groundwater in the deep aquifer below london, *CIRIA Special Publication 69*, Construction Industry Research and Information Association, London, 1989. Reproduced by kind permission of CIRIA: www.ciria.org.)

the 1990s. Abstraction lowered groundwater levels from almost ground level down to a 50-m depth, depleting groundwater resources and causing considerable ground settlement (Eddleston 1996). This resulted in the Thai government passing laws, for the first time, to regulate groundwater abstractions. Similar regulations exist in many other countries.

The protection of groundwater quality is important, because if groundwater becomes polluted, it can be very difficult to rehabilitate. It is therefore better to avoid or reduce the risk of contamination than to deal with its consequences. Groundwater quality is protected by the imposition of controls on discharges to the water environment (groundwater and surface water). This approach sets allowable limits on the concentrations of many dissolved or suspended substances in discharge water.

17.4.2 Abstraction of groundwater

The abstraction of groundwater for beneficial use or supply in England and Wales is restricted by law. The relevant legislation is the *Water Resources Act 1991*, which replaced earlier legislation dating from 1945 and 1963. This means that an abstraction license is required if groundwater is to be pumped for subsequent use, for example, by a farmer or by a factory (although low-volume abstractions for domestic use do not require a license). The license sets limits on the quantity to be pumped, and the requirement to obtain a license ensures that the regulator can keep records of total abstractions from particular aquifers.

The regulator responsible for the protection of groundwater resources in England and Wales, the Environment Agency (EA), considers the merits of the application in relation to groundwater resources in the area affected. Applications that raise significant environmental concerns or that are in areas where groundwater resources are overutilized may be rejected. Abstractions in Scotland and Northern Ireland are controlled by similar licensing systems.

The water abstraction licensing system set out in the *Water Resources Act 1991* provides a small number of exemptions, which can allow groundwater to be abstracted without the need for a license or consent. These exemptions apply if the total abstracted volume is very low or if the water is being abstracted for one of the purposes exempt under the Act, such as domestic use.

Traditionally, dewatering abstractions (for construction, mining, or quarry activities) were exempt from the licensing process, although the EA did have indirect powers to set limits on rates of abstraction and/or to require environmental mitigation measures.

This historic exemption from licensing likely developed, because in practice, for the great majority of construction dewatering systems, any impacts outside the immediate area of the works were minimal. This is probably because the rate of abstraction from dewatering pumping was a tiny fraction of the recharge available to the aquifer, or the area of aquifer dammed by a cutoff wall was small in relation to the extent of the aquifer. However, in a small number of cases, often involving large-scale construction dewatering abstractions from highly permeable aquifers such as the chalk, significant lowering of groundwater levels has occurred over a very wide area, perhaps 1 km or more from the site being dewatered.

However, changes in European legislation have meant that any significant construction dewatering systems will be part of the abstraction licensing system. The *EU Water Framework Directive* (WFD), adopted by the European Union in 2000, is intended to establish a framework for the protection of surface and groundwater. In relation to groundwater, the WFD will promote long-term protection of groundwater quality (by preventing and remediating groundwater pollution) and groundwater quantity (by controlling abstraction volumes to prevent overexploitation of aquifers).

Cumulatively, it is possible that, on a local level, abstractions for dewatering may form a significant proportion of total groundwater abstractions. The management of water resources in line with the WFD will be much easier if dewatering abstractions are licensed in a similar way to other groundwater abstractions, and recent changes in legislation have facilitated this. In England and Wales, from 2012 dewatering abstractions of greater than 20 m^3/day will require licensing, as set out in the *Water Act 2003*.

As part of the process of obtaining a license from the regulatory bodies, there will be a likely requirement to assess the groundwater impacts (see Section 15.9). This process of impact assessment is sometimes known as a hydrogeological impact appraisal (Boak et al. 2007).

Similar regulatory regimes exist in the rest of the United Kingdom, where the regulators are the Scottish Environmental Protection Agency and the Environment Agency Northern Ireland.

17.4.3 Discharge of groundwater

Once abstracted from the ground, the pumped water must be disposed of (volumes generated are normally too great to be stored on-site). The abstracted water from a groundwater control system is legally classified as *trade effluent*. In urban areas, it may be possible to dispose of water into sewers or surface water drains; permission must be obtained from the water utility or its agents before this can be done. Such permissions normally take the form of trade effluent licenses, which may set limits on the quantity and quality of water that can be discharged. Charges (based on a cost per cubic meter) are generally levied for disposal of water in this way. For long-duration discharge of substantial quantities, these charges may add up to considerable sums.

However, in many cases, the abstracted water must be disposed of to surface water including rivers, lakes, and the sea or to groundwater (via recharge wells or trenches). All surface and groundwater are legally classified as *Controlled Waters* under the *Water Resources Act 1991*. In these cases, if the duration of discharge is more than 3 months, an Environmental Permit is required from the EA. It is a criminal offense to discharge poisonous, noxious, or polluting material into any controlled waters, either deliberately or accidentally. Polluting materials include silt, cement, concrete, oil, petroleum spirit, sewage, or other debris and waste materials. Measures of preventing the discharge of polluting materials are described in Section 15.8.

Even if the discharge flow is to be disposed of by artificial recharge (see Section 11.9) back into the same aquifer, an Environmental Permit is still required. This means that the recharge water quality will have to be monitored to ensure that it is within the quality limits set on the environmental permit.

17.4.4 Settlement resulting from abstraction of groundwater

The lowering of groundwater levels resulting from the abstraction of groundwater will cause an increase in vertical effective stress in a soil. This will inevitably lead to some ground settlement, possibly over a wide area. Occasionally, the settlements may be large enough to cause distortion or damage of nearby structures or services. Engineering mitigation techniques are available to minimize the extent and effects of these settlements. Section 15.4 describes these problems and their potential solutions.

There has been considerable litigation over many years regarding settlement damage that has been alleged to result from groundwater abstractions for water supply or dewatering purposes. The law related to ground settlements arising from groundwater abstraction is found in common law rights, which are based on the precedents of judgments laid down in previous cases in the civil courts.

According to a number of reviews of the legal issues (such as the work of Akroyd 1986 and Powrie 1990), it is well established in common law that landowners have no right of support from the water percolating in undefined channels beneath their land. Groundwater in almost all aquifers is effectively flowing in undefined channels and is therefore considered by common law to be a reservoir or source of supply, which is no one's property but from which everyone has the right to abstract as much as they wish, in so far as it is physically possible to do so.

The courts have found that, if groundwater abstraction causes settlement damage to a neighbor's property, there is no right of action against the abstractor. The courts have also found that, because there is no duty of care to avoid damage resulting from groundwater abstraction, there is also no cause of action under the law of nuisance or negligence.

Possible conditions where groundwater would not be considered to be in undefined channels include well-established subterranean solution channels known to exist in karst aquifers or water flowing in abandoned mine workings. From a legal point of view, if water is believed to be flowing in a defined underground channel, the presence of the channel must be known. It is not necessary for it to be revealed by excavation or exposure, but its presence must be a reasonable inference from the available information.

This means that, if dewatering abstractions pump clear groundwater (with no suspended solids) from most aquifers, there is no legal liability if settlement damage occurs, even if mitigation measures could have been employed to prevent damage. This position is quite different from settlement damage arising from excavation or tunneling, where the primary mechanism is the removal of the support from the soil and a legal liability exists.

However, for dewatering schemes, if the water pumped is not perfectly clear and if it contains suspended silt or sand particles, the support of the ground is being removed. Landowners do have the right to the support of the ground beneath their land. If the pumping of silt or sand led to settlement damage, there would be a legal right of action against the abstractor. A consequence of this is that all wells and sumps must be equipped with effective filters to prevent the flow of groundwater removing fine particles from the soil. In practice, this is most likely to be a problem with sump pumping systems, and these should be employed with caution if structures are present near the excavation.

Although there is no legal liability for settlement damage resulting from the abstraction of groundwater, in the great majority of projects, construction methods are designed to be sensitive to the risks of consequential damage. If there is supposedly a significant risk of settlement damage, the project client or designer should rightly require the use of settlement mitigation or avoidance measures (such as artificial recharge).

17.4.5 Environmental regulation of major projects

Very large projects are, in principle, subject to the same environmental constraints as smaller projects. However, the sheer scale and high-profile nature of the projects may result in a slightly different approach being taken. This can be illustrated by the case history of the Medway Tunnel, constructed in southern England in the 1990s (Lunniss 2000; Thorn 1996).

The tunnel was of immersed tube design and required significant dewatering to allow construction of a casting basin and cut-and-cover tunnel sections on either side of the river. At the planning stage, the dewatering was anticipated to require extensive pumping from the chalk aquifer, which provided public water supply from a number of nearby wells. There was concern that the dewatering would derogate these sources and that saline intrusion may be promoted by river dredging during construction.

As with many large infrastructure projects in the U.K., the construction of the tunnel was covered by a special Act of Parliament—in this case, the *Medway Tunnel Act 1990*. Such acts require concerned bodies to petition for their “rights” in the drawing up of the bill and request special protective provisions. The environmental regulator at that time—the National Rivers Authority (NRA), the predecessors of the EA—obtained, via the Act, protective measures as follows:

> The wardens (of the scheme) … shall so design and construct the tunnel as to ensure by all reasonably practicable means that saline or other

contaminating intrusion into water resources in underground strata does not occur by reason of such construction.

The construction contract gave the contractor the responsibility of developing a dewatering and monitoring scheme acceptable to the NRA. Detailed liaison followed, and a dewatering and monitoring strategy was developed; this strategy was robust enough to cope with an increase in anticipated pumping rates from around 250 L/s up to 400 L/s. The monitoring exercise was intensive. The data gathered in the initial stages of the project allowed a numerical model of the aquifer to be developed by the contractor to support their proposals for the later stages of dewatering.

Although the extent of monitoring and discussion with the NRA is considerably more extensive than is routine, this project highlights that any major dewatering works are likely to require significant ongoing liaison with the environmental regulators. It is not unusual for the planned dewatering works to be varied in light of the regulator's requirements.

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Chapter 18

The future—a personal perspective by Toby Roberts

18.1 INTRODUCTION

The basic laws of physics govern the hydromechanical performance of groundwater lowering equipment and it is difficult to see how there could be any fundamental improvement in these systems. However, developments in materials, equipment design, and technology lead to improvements in plant efficiency, reliability, and safety over time. The drivers for these changes are diverse, ranging from environmental and safety regulation to the needs of other industries whose technology has been borrowed and adapted for use in construction dewatering. In addition, changes in other construction processes can open an opportunity for widening the application of groundwater control techniques. History shows that any consideration of the future needs to be tempered by Amara's law, which states:

> We tend to overestimate the effect of a technology in the short run and underestimate the effect in the long run.

When I first started in the construction dewatering industry in the United Kingdom 30 years ago:

1. All pumps, pipework, and well screens were made of steel with virtually no plastic components or flexible pipes.
2. Most dewatering projects involved steel wellpoints installed manually; deep wells with submersible borehole pumps were cutting edge, and ejectors were found only in textbooks.
3. There was little constraint on the abstraction or discharge of groundwater in the United Kingdom, and there was no visible enforcement of regulations.
4. Monitoring of flow rates and drawdowns was an expensive luxury.
5. Contracts and project management were openly confrontational, with little consideration given to health, safety, or the environment.

6. The fax had just replaced the telex as the most rapid form of written communication.
7. If a staff member wanted to speak to someone on-site, he/she had to wait until the person came back to the site office, if the site office had a phone at all. Staff members on overseas sites in remote areas were effectively on their own, with the best form of communication being air mail.
8. Computers had entered the workplace but only for word processing by secretarial staff; it was more than 10 years before most engineers had a personal computer (PC) on their desk.

The earlier chapters have shown that much has changed. This chapter explores recent innovations, regulations, and technology advances and assesses the potential impact of these on the future of groundwater control techniques.

18.2 APPLICATIONS AND TECHNIQUES

The components of dewatering systems are largely borrowed from other industries, and it is their needs that are driving many of the improvements in plant and equipment: the water-supply industry for borehole pumps, well screens, and ejectors; the piling and grouting industry for drilling systems; and process engineering for centrifugal pumps, exhausters, valves, and pipe systems. Changes are incremental and generally lead to an improvement in efficiency and reliability and a corresponding reduction in cost.

These improvements are important, because groundwater lowering techniques compete in the marketplace with other civil engineering processes used to control groundwater, such as cutoff walls, grouting, and artificial ground freezing. However, of greater interest are developments that can widen the scope and application of dewatering systems. The main potential must lie in the control of pore water pressure in fine-grained low-permeability soils. These soils also present a challenge to other techniques such as grouting. The wider use of ejector systems in the United Kingdom over the last 20 years has extended the application of dewatering systems to fine-grained soils (Roberts et al. 2007). There is some evidence from detailed monitoring of dewatering systems in fine-grained soils that there can be a significant variation in the yield from individual wells, installed in a similar manner in apparently similar ground conditions. It is not clear if this is due to some installation effect or undetected variations in the soil fabric local to the wells, or both. There would be significant cost and performance benefits if a system could be devised to ensure that all the wells in a system were as efficient as the best performing well. Any solution will probably involve modification to some or all of the current methods of well drilling, well installation, screen specification, filter-pack specification/placement, and well development.

Another approach used to control pore pressure in fine-grained soils is electro-osmosis (see Section 11.8). This has been developed and used effectively, particularly in Canada, but has rarely been used in the last 30 years. Over the last few years, there has been renewed interest in electro-osmosis, in particular for recovering heavy-metal contaminants as a means of soil remediation. The technique is known as electrokinetic or electro-chemical remediation. A practical application of the technique is described by Trombly (1994). Studies have also been made into the use of electro-osmosis applied by conductive geosynthetics to dewater fine, high-water-content soils and waste sludge and to enhance band drain performance (Jones et al. 2005). Innovations such as these could herald a renaissance in electro-osmosis.

Siphon drains are described in Section 11.7. They are a rare example of a previously unused method of pumping being applied to groundwater control. The attributes of siphons are particularly suited to slope stability problems, which are often caused by excess pore water pressures. This is clearly a niche technique but offers considerable potential cost saving over a "hard" retaining wall or piled solution.

Changes in project requirements and other construction processes and methods can also offer opportunities:

1. *Access constraints*. Construction of new and improved infrastructure in cities must be carried out while minimizing disruption to existing services and infrastructure. This requirement often introduces significant access constraints (McNamara et al. 2008). Sometimes, constraints can be overcome by installing inclined or horizontal wells or wells that are installed directly out of tunnels. Hartwell (2001) describes techniques for drilling wells from tunnels, but there remains significant room for improvement.
2. *Directional drilling and horizontal wells*. Directional drilling techniques were developed to allow pipelines and services to be installed without trench works, but these techniques have also been used to install horizontal wells (see Section 11.4). The majority of applications to date have been for water supply and ground remediation, but long-term dewatering schemes using horizontal wells drilled from collector shafts are in use to prevent groundwater and sand ingress at Govan Station on the Glasgow subway (see Section 14.7) and to mitigate rises in groundwater levels as a result of the Cardiff Bay Barrage (Williams 2008). Costs are significant, the installation of filter packs is challenging, and development procedures are not established. Nevertheless, there are clear opportunities where surface access is constrained.

3. *Compressed air working in tunnels*. Health and safety regulations mean that there are significant cost and health risks associated with working in air pressures above 1 bar. In cases where pressures above 1 bar would normally be required, it has sometimes been proven cost-effective to use a dewatering system to lower pore pressures so that working air pressures can be reduced to below 1 bar.
4. *Pressure relief on retaining walls*. Controlling pore water pressures can reduce the need for temporary propping or anchoring of retaining walls. Controlling pore water pressures on the external "active" side will reduce loading. Lowering groundwater levels on the internal "passive" side to significantly below excavation level can improve the soil properties and increase the passive resistance.
5. *Soil remediation*. An effective method of removing volatile contaminants from soils is by soil vapor extraction. A high water table prevents movement of vapor and curtails the effectiveness of this technique. Dewatering systems can be used to reduce the water level to improve the performance and reach of the remediation process.
6. *Landfill leachate*. New engineered landfills have built-in leachate drainage systems. However, there are many older landfills where retrofitting of a leachate drainage system is needed. The combination of aggressive chemical conditions, substantial settlement, and usually low permeability makes this a demanding environment for dewatering systems. Well systems are used with some success, but there is scope for improving well design and the reliability of the pumping systems.
7. *Pile construction*. Bored piles are often installed in unstable soils below the water table using bentonite slurry to provide temporary borehole support during pile construction. Reducing groundwater levels to below the toe of the piles can have a number of benefits: less bentonite is used, the bentonite does not require desanding, the pile installation time is some 30% faster, and the load capacity of the finished pile is greater. Although substantial drawdowns may be needed, for a large project the savings in cost and time can be significant.
8. *Sprayed concrete lining (SCL) tunneling*. The SCL tunneling method can be an efficient method of forming irregularly shaped caverns underground (e.g., for underground stations during the construction of metro systems). The technique relies on short advances of excavated soil to be self-supporting for a few hours while the sprayed concrete is applied and sets. This is not a problem in weak rock and stiff clays but is an issue in water-bearing granular soils. Pore water pressure control using wells from the surface or wellpoints installed from the heading can sometimes be used to extend the application of SCL techniques into otherwise marginal soils.

9. *Open-loop geothermal systems.* Open-loop geothermal systems are an environmentally attractive method of providing heating, ventilation, and air-conditioning (Banks 2008). "Total loss" ground source heating and cooling systems, where all the abstracted water is discharged to waste, are not generally environmentally acceptable due to the waste of water. As a result, most systems comprise the abstraction and artificial recharge of groundwater far enough apart to maintain hydraulic and thermal efficiency. Occasionally, temporary dewatering wells have been converted for use as open-loop geothermal systems.

These are some examples of industry demands and innovations that can widen the scope and application of groundwater control systems, and the ingenuity of engineers will doubtlessly find more opportunities in the future.

18.3 COMMUNICATION AND MONITORING TECHNOLOGY

There have been significant improvements in monitoring technology, while the rapid advance of mobile communications continues to impact all areas of our working and private lives. High-definition videophones, videoconferencing, and remote-site viewing technology systems are available now but will become increasingly common as costs fall and the technology becomes more familiar and easier to use. These forms of communication, combined with the increased use of automated monitoring systems, can give near-real-time performance data to anyone connected to the Internet. Large items of construction plant are generally already Internet enabled, allowing remote performance monitoring, and this technology will become increasingly available for smaller items of equipment, including individual pumps, in the future. These innovations will be important for all aspects of construction, particularly for the delivery of dewatering services, which requires the application of experience and engineering judgment based on feedback from performance monitoring, together with reliable plant operation and continuous running 7 days per week.

Advances in communication have had a profound impact on my working life, and it is clear that widespread adoption of the technologies currently available, let alone new ones, will bring substantial changes in the future. The pace of change is moderated only by a lack of common standards, including effective methods for visualizing large, complex datasets, and by our own ability to absorb and respond to information. The datasets obtained for individual projects together with back-analysis may be used to increase our understanding of the flow regime in the vicinity of well arrays, leading to improvements in design and modeling software.

18.4 NUMERICAL MODELING

The penetration of computer technology into the world of construction dewatering in the United Kingdom started with word processing in about 1980, progressed to cost estimating and accounts, before reaching the desks of design engineers around 1990. Dataloggers (initially desktop PCs with appropriate software and sensors) were used on-site for monitoring occasional high-specification pumping tests as early as the mid-1980s but were not in regular use until the mid-1990s. By 2000, datalogger systems were being widely used to provide cost-effective remote automated monitoring and alarms for operating dewatering systems. Remote plant operation and control systems are available but are not in regular use in 2012. Despite these developments, it remains the case today that the vast majority of small- and medium-sized dewatering schemes, with a straightforward geological setting, are designed on the basis of experience, local knowledge, and empirical "rules of thumb." Two-dimensional or radial steady-state analyses (see Chapter 7) may be undertaken to justify the design. Finite-element numerical modeling may be used to look at the regional impact of a large scheme. Numerical modeling is also used when the geometry of a scheme is not amenable to analysis using closed-form solutions for plane or radial flow, as in the case of flow under a partially penetrating cutoff. Current groundwater numerical models are immensely sophisticated and can account for both unsaturated flow and negative pore pressures in two, three, and four dimensions (i.e., with time-dependent effects). These programs are moving beyond mere modeling toward "virtual reality." Clever interfaces and element building tools allow models to be put together quickly. However, three-dimensional time-dependent modeling remains too time consuming for most medium-sized and smaller projects and, in any case, is often not justified. A further shortcoming of three-dimensional models is the inability to realistically model conditions in the immediate vicinity of a well. This is generally overcome by making simplifying assumptions, such as combining several wells into one sink and ignoring any seepage face at the wells. For the purposes of assessing total seepage flows to an excavation, these simplifications are reasonable, and the resulting errors in flow estimates are generally small. However, such simplifications effectively prevent direct assessment of the number, size, and depth of dewatering wells required. It is clear that modeling software will need to continue to improve in the future.

A numerical model is much less costly than a fully instrumented field test, and modeling software has been exploited to some extent as a way of developing accessible design tables or charts for particular geometries or conditions. Good examples are the design charts given by Powrie and Preene (1992), which bridge the gap between radial analysis and two-dimensional plan analysis. A fruitful area of further study would be to establish the scale of the errors resulting from applying two-dimensional or radial analysis to problems that are strictly three-dimensional. This would put the relatively simple two-dimensional and radial analysis on a firmer footing and make it clearer where a three-dimensional model is needed.

Numerical modeling techniques are having an important impact on the design of groundwater control systems, but they remain a long way from challenging the central role of judgment based on experience, local knowledge, and empirical "rules of thumb" in the design process. In the meantime, modeling will continue to be used to support design assumptions (principally for parametric analysis), to examine environmental and settlement risks, and to back-analyze, visualize, and interpret monitoring data obtained during the works.

18.5 REGULATION

When reviewing this chapter for the second edition, I was surprised to find that this section on future regulation was already history. Although Europe is at the forefront of effective regulation and enforcement, some form of regulation is now evident in all of the jurisdictions in which I have worked over the last 10 years (United Kingdom, Ireland, United Arab Emirates, Oman, Qatar, and Hong Kong). The following requirements now tend to be ubiquitous:

1. Discharge consent, which generally includes controls on flow rate and quality (notably suspended solids and contaminants). Monitoring and reporting of flow and quality is generally obligatory, with spot-checks by the authorities not uncommon.
2. Consent for artificial recharge back to the ground (including shallow soakaway pits), which is generally considered a form of discharge.
3. Abstraction consent, which, in areas where groundwater is not considered a resource, may be included with the discharge consent or excavation planning consent.
4. General construction health and safety regulations to protect site workers and the public.

The issue of discharge can be important and, in some cases, be a "show stopper" where there is no access to open water or a storm sewer. Installing discharge mains across private land has to be agreed with individual site owners. Such way leave agreements can prove difficult or impossible to obtain where a landowner has no interest in supporting the completion of a project and demands excessive recompense.

In addition to these requirements, there may be local controls and regulation, for example:

1. Central Copenhagen and Amsterdam have blanket bans on groundwater lowering. Many of the old buildings in these cities are founded on weak soils, often with wooden foundations, and experience has shown that consolidation and rotting of the wooden foundations occurs in the event of even small reductions in groundwater level.
2. Groundwater is an important resource in Berlin. In order to protect it from wastage, dewatering systems are licensed with a fixed extraction volume. Oftentimes, this can only be met by extensive use of cutoff walls or very rapid construction.
3. Wells below a certain depth in Denmark must be installed by a suitably trained driller. This is to ensure that the driller fully understands the need for appropriate annular seals to protect water-supply aquifers from saline or polluted water in the superficial shallow aquifers.

It is worth noting that none of the above local regulations has prevented significant groundwater control exercises in these areas; indeed, in the last decade, major excavations have been carried out successfully for metro construction in Amsterdam and Copenhagen (Bock and Markussen 2007). In fact, what has happened is that the regulatory constraints have driven innovation in dewatering and ground treatment on those projects.

These types of regulation, which generally are a response to local conditions and experience, are likely to become more common in the future. Care and research is required to ensure that the requirements are identified and fully understood.

One implication of these regulations is that a good understanding of the groundwater regime is generally required in order to develop schemes to meet the imposed constraints. This can only be achieved with high-quality geotechnical investigations, including site-specific pumping tests combined with relevant experience. Oftentimes, a sophisticated conceptual model of the ground, together with numerical groundwater modeling, will also be needed in order to obtain permission to proceed with the works. For major projects, at least, this can go some way toward addressing one of the geotechnical community's oldest laments, that is, site investigations are too often inadequate (Institution of Civil Engineers 1991).

The issues outlined above are direct controls on dewatering activities, but other forms of legislation and regulation can prove to be significant. Recent examples include:

1. The *Provision and Use of Work Equipment Regulations*, which were originally introduced in the United Kingdom in 1992 and have been amended and updated several times since (see Section 17.2). The

UK Health and Safety Executive began enforcing the requirement for guarding rotating parts with respect to site plant in 2005, and today, all site drilling rigs in the United Kingdom must comply with these regulations. The long working life of a drill rig, combined with the fact that the United Kingdom is currently one of the few countries to enforce these EU regulations, means that retrofitting of guards is commonplace. One practical effect is to limit how close to a vertical wall a well (or pile) can be installed. Although it is probable that rig manufacturers will find solutions to resolve the current unsatisfactory situation, the impact on construction work is very real in the meantime.

2. The *EU 2005 EcoDesign Directive* for electric motors has been in force since 2011. This requires all electric motors sold in the European Union to meet strict efficiency criteria. The potential savings on power consumption for electric pumps are significant. For the moment, these regulations do not apply to submersible electric motors, but this would clearly be a logical future extension. The precise way in which these regulations may affect pump design is unclear, but any impact on the diameter of borehole pumps could have profound implications for dewatering operations, which routinely work to tight tolerances on well screen sizes and pump diameters.

It might be thought that environmental legislation could represent a threat to the future use of dewatering systems, but compared to alternative processes, the environmental impact of groundwater lowering systems is relatively benign. The energy consumption for dewatering systems is modest, the impact on groundwater flows should be temporary, and only the well screens are left behind on completion (even these could probably be removed with a bit of ingenuity). Well screens are routinely sealed to avoid the risk of cross-links between aquifers. In comparison, artificial ground freezing has a heavy energy draw and uses refrigerants with questionable environmental credentials. Grouting involves pumping potentially large quantities of environmentally unfriendly material into the ground and leaving it for all time. Tools are being developed for assessing the carbon footprint and wider sustainability of construction activities and materials (Hughes et al. 2011). Some project clients are now taking a lead in demanding an assessment of the embedded carbon in construction works, and it is probable that in due course, legislation will require engineers to assess alternative construction techniques.

18.6 WHERE DO WE GO FROM HERE?

In this chapter, I have drawn attention to the areas of innovation and regulation, that I anticipate will be the main drivers for future changes. Amara's law highlights the difficulty of assessing the short-term versus long-term impact of changes in technology. Nevertheless, innovation in construction generally moves forward at a stately pace, and as a result, I anticipate that changes in communication technology, regulation, and materials will have the most significant impact on the application of groundwater control techniques over the next 10 years.

18.7 NEXT GENERATION OF DEWATERING PRACTITIONERS

As this book shows, the successful application of groundwater control schemes requires a sound understanding of engineering and hydrogeology, together with a wealth of experience. The value of this book is that it represents the distillation of more than a lifetime of experience by two engineers whom I greatly respect. However, we sometimes fail to recognize and appreciate the vital contribution made by the site managers, site supervisors, drillers, and site operatives who install and operate these systems and can rightly claim more raw practical experience than any design engineer. There are few shortcuts to gaining experience, and I worry about the loss to our industry each time one of our site staff moves on or retires. We need to identify and develop the next generation of dewatering specialists and equip them with the necessary skills, both theoretical and practical, to allow them to take dewatering technology and practice forward over the coming decades.

Appendix 1: Estimation of permeability from laboratory data—Loudon method

Loudon (1952) proposed an empirical formula, based on earlier work by Kozeny, to estimate the permeability of granular material:

$$\log_{10}(kS^2) = 1.365 + 5.15n \tag{A1.1}$$

where k is the permeability expressed in centimeters per second, n is the porosity of granular soil (a dimensionless ratio expressed as a fraction, not as a percentage), and S is the specific surface of grains (surface area per unit volume of grains) expressed in square centimeters per cubic centimeter. Note that the values in the formula are specific to these units. Care must be taken when applying this method to avoid introducing errors by using different units.

A1.1 ESTIMATION OF SPECIFIC SURFACE *S*

According to Loudon, the specific surface S_i of spheres uniformly distributed in size between mesh sizes D_x and D_y of adjacent sieves is given by

$$S_i = \frac{6}{\sqrt{D_x D_y}} \tag{A1.2}$$

which is accurate to within 2% if (D_x/D_y) is not greater than two. The specific surfaces of spheres lying between given grain sizes are set out in Table A1.1 based on data from the work of Loudon (1952).

Of course, very few actual soils have grains that are even approximately spherical. The actual specific surface S of a soil is influenced by the angularity factor f (also known as the coefficient of rugosity). The specific surface S of nonspherical grains is equivalent to the specific surface of spheres S_i multiplied by the angularity factor f.

The more angular the shape of the grains is, the greater the specific surface will be. According to Loudon, a visual estimate (using a hand lens) of

Table A1.1 Specific surface of spheres lying between given sieve sizes

British standard sieve size	*Sieve size (mm)*	*Specific surface (cm^2/mm^3)*
7–14	2.4–1.2	35.2
14–25	1.2–0.6	70.6
25–52	0.6–0.3	143.0
52–100	0.3–0·15	284.0
100–200	0.15–0.075	558.0
7–10	2.4–1.9	29.6
10–14	1.9–1.2	41.9
14–18	1.2–0.8	59.2
18–25	0.8–0.6	83.8
25–36	0.6–0.45	119.0
36–52	0.45–0.3	170.0
52–72	0.3–0.2	242.0
72–100	0.2–0.15	336.0
100–150	0.15–0.1	476.0
150–200	0.1–0.075	675.0

the angularity factor seems quite good enough for the standard of accuracy generally needed. He stated that f may be estimated roughly as

Material type	*Angularity factor f*
Glass beads	$f = 1.0$
Rounded sand	$f = 1.1$
Sand of medium angularity	$f = 1.25$
Angular sand	$f = 1.4$

Thus, in practice, the range in angularity factor of natural soils is not great, being approximately 1.1 to 1.4—a full range variation for naturally occurring soils of about 25%. Loudon further indicated that very angular materials, such as crushed marble, crushed quartzite, or crushed basalt, may have angularity factors of 1.5 to 1.7.

Once a particle size test has been carried out using sieve of various sizes, the specific surface of a sand sample can be estimated by

$$S = (x_1S_1 + x_2S_2 + \ldots + x_nS_n) \tag{A1.3}$$

where $x_1 \ldots x_n$ are the fractions of the total mass (i.e., $x_1 + x_2 + \ldots x_n = 1$) retained on different sieves, and $S_1 \ldots S_n$ are the specific surfaces of spheres uniformly distributed within the corresponding sieved fractions, as listed in Table A1.1.

Table A1.2 In situ porosity and void ratio of typical soils in natural state

Description	*Porosity (n)*	*Void ratio (e)*
Uniform sand, loose	0.46	0.85
Uniform sand, dense	0.34	0.51
Mixed-grain sand, loose	0.40	0.67
Mixed-grain sand, dense	0.30	0.43

Source: Terzaghi, K., Peck, R.B., and Mesri, G., *Soil Mechanics in Engineering Practice*, 3rd edition. Wiley, New York, 1996.

A1.2 ESTIMATION OF POROSITY *n*

The other relevant factor is in situ porosity n. The range of values for granular sands to which Loudon's method applies is from approximately 0.26 (densest state) to approximately 0.47 (loosest state), although mostly in the range of 0.30 to 0.46—quite a narrow range variation. Some values of porosity of typical soils are summarized in Table A1.2.

Although the porosity of a sample can be determined in the laboratory, it is virtually impossible to determine the in situ porosity of a sample. The porosity should be estimated from published values (such as from Table A1.2), appropriate to the soil description. This is a limitation on the usefulness of Loudon and other similar works and an explanation for the somewhat erratic results that they sometimes give.

A1.3 ESTIMATION OF PERMEABILITY

Once porosity n (as a fraction) has been estimated from Table A1.2 and specific surface S (in square centimeters/cubic centimeters) has been calculated using Equation A1.3 and Table A1.2, Loudon's formula can be used to calculate permeability k as

$$k = \frac{10^{(1.365+5.15n)}}{S^2} \tag{A1.4}$$

Permeability is calculated in centimeters per second. To convert to meters per second, the k value in Equation A1.4 should be multiplied by 0.01.

Like many formulas for estimating permeability from particle size data, Loudon's formula is only valid for uniform sands between specific particle size ranges (in this case, approximately between 1 and 0.075 mm). Loudon notes that the calculated permeability became unreliable when the soil contained more than 5% of particles finer than 0.075 mm.

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Appendix 2: Execution and analysis of variable head permeability tests in boreholes

A2.1 GENERAL COMMENTS ON PERFORMING PERMEABILITY TESTS

Many factors may influence the results obtained from in situ permeability tests. Great care must be exercised to ensure that tests are carried out under controlled conditions; some tests are reliable only when performed according to specific conditions and these should be strictly adhered to. Some general guidance is given as follows:

1. When making tests in boreholes, before the test, the borehole should be carefully cleaned out to remove loose or disturbed material, the presence of which can introduce significant errors in the results obtained. Any sediment-laden water in the borehole should be flushed out and replaced with clean water.
2. When carrying out inflow tests, the water used should be clean, and attention should be given to the cleanliness of tanks, buckets, and pumps so that the water does not become contaminated by dirt in the equipment, because very small amounts of silt may cause serious errors in the results.
3. The water level should be measured with a dip meter relative to a stable datum such as the top of the borehole casing or a firmly positioned rod across the top of the borehole.
4. In the early stages of rising and falling head tests, it is desirable to take readings of the water level at frequent intervals of about 30 s. To obtain reasonable accuracy in these circumstances, it is necessary to have two persons taking the readings: one to adjust the dip meter to keep pace with the change in water level and to take the depth readings and the other to call out the time intervals from a stopwatch and record the water levels. Alternatively, a pressure transducer linked to a stand-alone data logger (see Section 16.6) may be installed in the borehole and programmed to take pressure readings every few seconds.

5. The equations for calculating permeability are all based on the assumption that flow through the tested material is laminar. To be sure of laminar flow, velocity should not exceed 0.03 m/s. Therefore, the tests are only applicable when the flow rate (into or out of the borehole) divided by the area of test section is less than 0.03 m/s. The open area is taken as the peripheral area of the filter section around a well or the intake area of the sides and base in the case of an unsupported hole.
6. Tests should, if possible, be made during dry weather. If a test has to be made while it is raining, measures must be taken to prevent surface runoff from entering the test borehole. If the borehole casing is close to ground level, this may involve using sand bags to build a small dam around the top of the borehole.

Where the test is carried out in an unconfined aquifer and surface conditions are relatively pervious, rainfall during the test may produce a change in the groundwater level, which could affect the results obtained. In these circumstances, careful note should be made of any showers immediately preceding or during a test. The groundwater level should be measured before and after the test.

A2.2 VARIABLE HEAD INFLOW (FALLING HEAD) TEST—TEST PROCEDURE

Each test is carried out at a specific depth interval known as the "test section" (Figure A2.1). The entire test section must be below the groundwater level. The initial groundwater level should be measured before making the test. It is important that the "pretest" groundwater level is representative of the natural, background, groundwater level. If the water level has been influenced by drilling or other activities, it will be necessary to wait until water levels have recovered to their original levels.

Before the test itself, the borehole is advanced to the base of the test section. Where casing is not necessary to support the borehole, it must nevertheless be inserted to the top of the test section so that only the test section is exposed to the strata. The casing must be tight against the side of the borehole; otherwise, the test water will leak around the casing. This test is suitable for testing at various depths during the progress of boring, where the test section will stand unsupported.

If casing is required to support the sides of the hole, it must be taken down initially to the base of the test section. Fine gravel backfill is then placed in the borehole while withdrawing the casing to expose the required length of unlined hole for the test; subsequently, the gravel fill should be brought up with and always kept just above the bottom of the casing as this

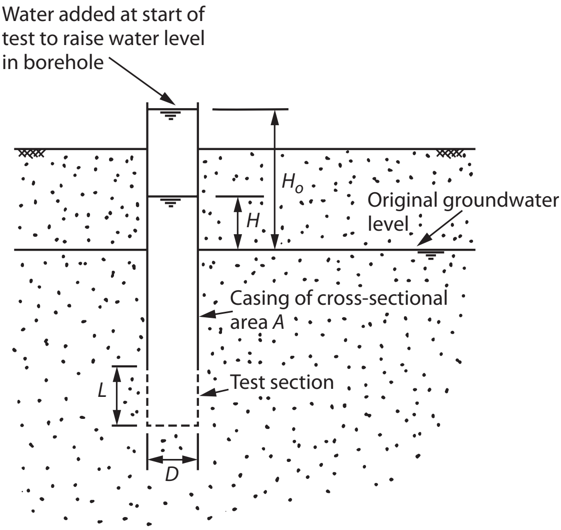

Figure A2.1 General arrangement of variable head tests in boreholes (falling head test shown).

is withdrawn (Figure A2.1). It is good practice, before placing the gravel, to replace any water standing in the borehole with clean water; this helps avoid suspended silt becoming trapped in the test section, which would reduce permeability. This type of test can also be used at various depths during boring, but the gravel backfill will need to be drilled out after each test; this will slow down the drilling progress.

It is best if these tests are carried out at the start of the day's work, as the original groundwater level will be less affected by drilling activities. If carried out during the day's drilling, the original water level recorded may not be the "true" natural water level. If necessary, the start of the test should be delayed until readings show the pretest groundwater level has stabilized.

To start the test, clean water is rapidly added to the borehole to raise the water level as high as possible in the casing; no more water is added during the test. It is essential that the water is added as rapidly as possible; the analysis methods assume that the initial change in head in the borehole is effectively instantaneous.

Once the addition of water has ceased, readings of the water level inside the borehole are taken at frequent time intervals, taking note of the time of corresponding to each reading. Time t = 0 is taken as the time of the first reading. No fixed time intervals can be specified, as the frequency of readings will depend on the rate of fall of water level. As a guide, an attempt should be made to take the readings so that each level differs from the previous reading by about equal increments. Because the water level will fall most rapidly at the start of the test and then at a decreasing rate, readings will become less frequent as the test progresses. If the initial head of water in the borehole above original groundwater level was, for example, 2 m, take readings so that the difference in successive readings is about 100 mm; if the head is smaller, the difference in level readings should be smaller, for example, 25–50 mm for the initial head of 0.5 to 1 m. Ideally, readings should be continued until the head of water in the borehole above groundwater level is less than one-fifth the initial head above the original groundwater level.

The following information should be recorded:

1. Diameter of unlined borehole being tested and diameter of casing
2. Depth to the base of the borehole (bottom of test section)
3. Depth to bottom of casing (top of test section)
4. Depth to top of gravel backfill, if used
5. Date and time that water level readings are started
6. Water level readings and time of each reading
7. Depth to the original groundwater level

All depth and water level readings should be measured relative to a clearly defined datum, ideally at ground level. If the ground level is uneven or flooded, the top of the casing can be used as a datum, provided that the casing is secured so that it will not slip or sink during the test.

A2.3 VARIABLE HEAD OUTFLOW (RISING HEAD) TEST—TEST PROCEDURE

This test should be used only in lined boreholes or in observation wells, provided that bailing or pumping-out is practicable. Each test is carried out at a specific depth interval known as the test section. The entire test section must be below the groundwater level; ideally, the borehole should penetrate below groundwater level by at least 10 times its diameter. The initial groundwater level should be measured before initiating the test.

Preparation of the borehole and test section is similar to that for an inflow test. At the start of the test, water is rapidly bailed or pumped out of the borehole to just above the bottom of the casing. No more water is removed during the test. It is essential that the water is removed as rapidly as possible; the analysis methods assume that the initial change in head in the borehole is effectively instantaneous.

Once water removal has ceased, readings of the water level inside the borehole are taken at frequent time intervals, taking note of the time corresponding to each reading. Time $t = 0$ is taken as the time of the first reading. Readings should be taken at the same general intervals as an inflow test and the same data recorded. Readings should be continued until the difference between the water level in the borehole and original groundwater level is less than one-fifth the initial difference in these levels.

It is best if these tests are carried out at the start of the day's work, as the original groundwater level will be less affected by drilling activities. If carried out during the day's drilling, the original water level recorded may not be the true natural water level.

A2.4 VARIABLE HEAD (FALLING AND RISING HEAD) TESTS—CALCULATION OF RESULTS

The permeability k may be determined for a variable head test using the following formula:

$$k = \frac{A}{FT} \tag{A2.1}$$

where A is the cross-sectional area of the borehole casing (at the water levels during the test), T is the basic time lag, and F is a shape factor dependent on the geometry of the test section.

A2.5 DETERMINATION OF SHAPE FACTOR *F*

The geometry of the test should be compared with the analytical solutions of Hvorslev, given in Figure 6.10, and the appropriate shape factor should be calculated.

A2.6 DETERMINATION OF BASIC TIME LAG *T*

1. Where the original groundwater level is reliably known, plot values of H/H_o on a logarithmic scale against corresponding values of elapsed time t on an arithmetic scale (Figure A2.2). H_o is defined as the excess head of water (measured relative to the original water level) at the start of the test (the initial reading at t = 0). H is the excess head of water at time t during the test. Draw the best fitting straight line through the experimental points. In some cases, the experimental points for values of H/H_o near 1.0 may follow a curve; these should be disregarded, and the straight line is drawn through the remaining points; then, draw a parallel straight line through the origin. The basic time lag T is obtained by reading off the value of t when H/H_o = 0.37 using the straight line through the origin.

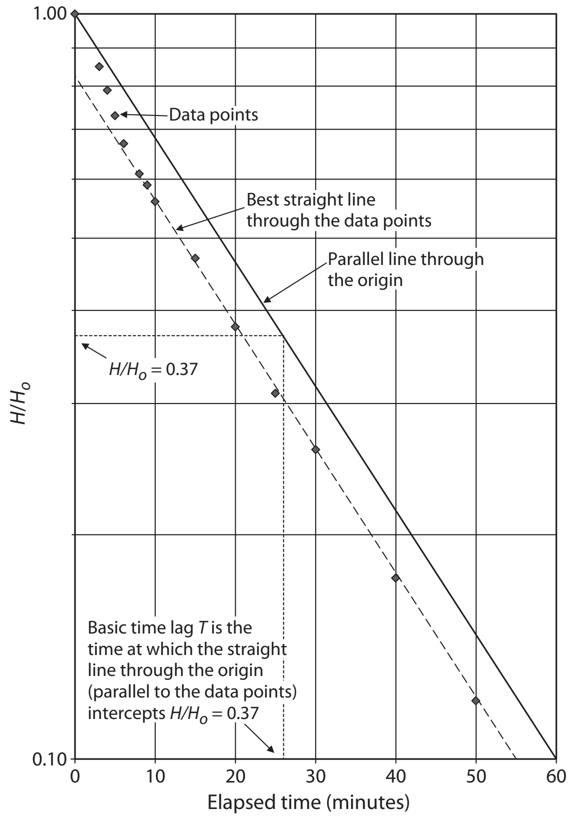

Figure A2.2 Estimation of basic time lag (original water level known).

2. If the original groundwater level is only approximately known, calculate values of *H* from an assumed groundwater level (making as accurate an estimate as is possible). Plot the resulting values of H/H_o on a logarithmic scale against corresponding values of *t* on an arithmetic scale (Figure A2.3). If the slope of the line through these points decreases with increasing *t* (curve A), the assumed original groundwater level was too low. If the slope increases with increasing *t*, the assumed groundwater level was too high. By trial and error, a groundwater level can be determined from which values of H/H_o plot as a straight line against *t*, at least for the higher values of *t*. Experimental points for small values of *t* may follow a short curve because of the flow settling down into equilibrium with the soil. Basic time lag *T* can then be determined in the same way as for step 1.

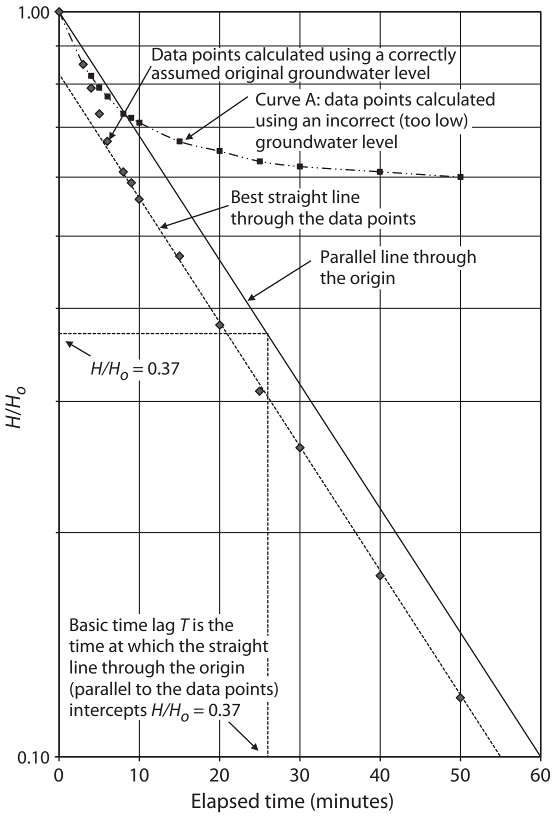

Figure A2.3 Estimation of basic time lag (original water level unknown).

The values of A (in square meters), F (in meters), and T (in seconds) are applied in Equation A2.1 to produce the calculated permeability k (in meters per second).

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Appendix 3: Execution of well pumping tests

A3.1 PLANNING OF PUMPING TESTS

The simplest form of pumping test involves controlled pumping from a well—the "test well"—and monitoring of the discharge flow rate from the well and the drawdown in observation wells at varying radial distances away (Figure A3.1). A pumping test is one of the more expensive in situ tests and requires careful planning to ensure that time, effort, and money are not wasted.

A pumping test consists of a number of phases:

1. *Prepumping monitoring*. This involves monitoring natural groundwater levels in the observation wells (and test well, if possible) for a period of a few days to several weeks before commencing pumping. The aim is to determine any natural or artificial variations in groundwater level that may affect the drawdowns observed during pumping.
2. *Equipment test*. This is a short period of pumping (typically 15–120 min) to allow correct operation to be ascertained of the pumps, flowmeters, and dataloggers and to check for leaks in the discharge pipework. It is normal to take some crude readings of flow rate and drawdown in the well to allow the selection of flow rate for subsequent phases.
3. *Step–drawdown test*. This is a period of continuous pumping typically lasting 4–8 h, during which the flow rate is increased in a series of steps. Each step is of equal duration (normally 60 or 100 min) and an appropriately designed test should have four or five steps at roughly equal intervals of flow rate. Water levels are normally allowed to recover for at least 12 h before the constant rate pumping phase can be started.
4. *Constant rate pumping phase*. This is often considered to be the main part of the test and involves pumping the well at a constant flow rate (chosen following the step–drawdown test). The constant rate test normally lasts between 1 and 7 days, although test durations of up to 28 days are not unknown.

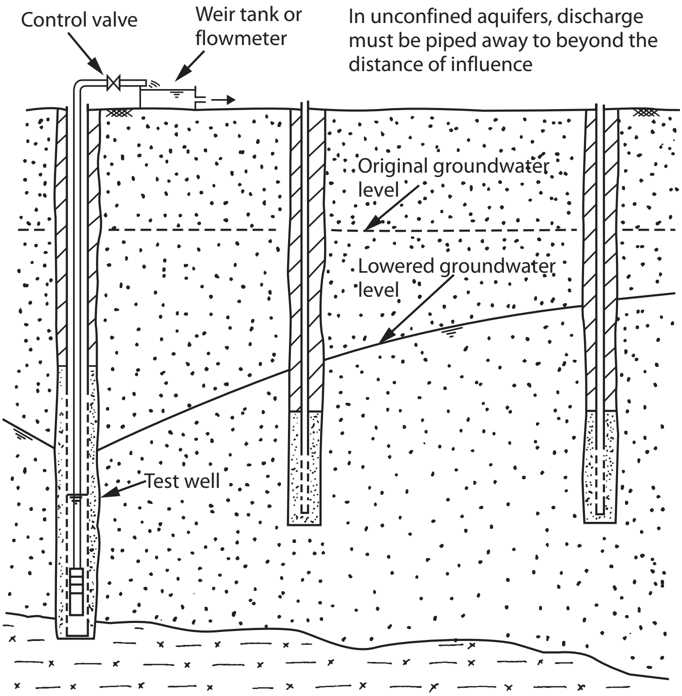

Figure A3.1 Components of a pumping test.

5. *Recovery phase.* Water levels in the observation well and test well are monitored as they recover after cessation of pumping. This phase often lasts between 1 and 3 days.

Guidance on the execution of pumping tests is given in BS ISO 14686:2003 and the work of Kruseman and De Ridder (1990).

Pumping tests as part of groundwater lowering investigations need to be designed to meet certain criteria consistent with providing high-quality data to the dewatering designer:

1. The aim should be to achieve drawdowns (measured in piezometers within 10–20 m of the test well) of at least 10% of the required drawdown in the proposed groundwater lowering scheme.
2. If the site is subject to natural groundwater level variations (such as tidal effects), the test should aim to achieve drawdowns significantly greater than the background variations.
3. It is essential that pumping is absolutely continuous during the initial hours of pumping. If pumping is interrupted due to whatever cause, such as a pump or power failure, it is essential that the test is

suspended until groundwater levels return to equilibrium, or those levels recorded before the commencement of any pumping. Only then should pumping recommence. Once the test pumping has being going on for 24 h or more, occasional short interruptions (for example, for daily checks of generator oil levels) are permissible. These interruptions should be as short as possible (ideally, no longer than a few minutes); any longer interruptions may require the test to be suspended and restarted later. Ideally, pumping should continue until steady state drawdown conditions are established in observation wells near the test well.

4. Most of the simpler methods used to analyze pumping test data assume that the well fully penetrates the aquifer. Ideally, the test well should be fully penetrating. However, provided the screened length is on the order of 70% to 80% of the aquifer thickness, the accuracy of the assessment of permeability is generally acceptable. Where the amount of penetration is lower, the flow will not be predominantly horizontal, and more special corrections may need to be applied to the drawdown data.
5. Sufficient observation wells should be installed, in suitable locations, to allow groundwater flow patterns to be identified. Ideally, there should be least three lines of observation wells radiating out from the test well, with each line spaced 120° apart. These need not penetrate the full depth of the aquifer but must be deeper than the expected drawdown (an observation well that becomes dry is not acceptable.). Each line should consist of three or more observation wells. Depending on the permeability of the aquifer, the distance from the test well to the nearest observation well should be in the range of 2–3 m (in a low to moderate permeability aquifer) or to 5–10 m (in a high-permeability aquifer) from the test well. More distant observation wells should be located within the anticipated distance of influence of the test well.

A3.2 GENERAL COMMENTS ON PERFORMING PUMPING TESTS

The test well should be designed, installed, and developed according to the guidelines in Chapter 10, and observation wells should ideally be consistent with the good practices outlined in Section 6.6. The most common form of test well is a deep well pumped by a borehole electro-submersible pump. Suction pumps can be used, but only in highly permeable soils with a high water table; otherwise, as drawdown occurs, the pump may become starved of water, giving a varying flow rate. Regardless of the method of pumping used, it is desirable to measure the water inside the test well during pumping. This will require the installation of a dip tube in the well so that the dip meter tape does not become entwined around the pump riser pipe due to the swirl of water near the pump intake.

The principal parameters to be measured during a test are water levels in the test well and observation wells, and the discharge flow rate from the well. All readings need to be referenced to the time elapsed since the start of pumping (or recovery) for that test phase. The frequency of monitoring must be matched to the rate of change of the measured parameter. During a test, the flow rate should remain approximately constant, but the water levels will fall rapidly immediately after the start of pumping, and will fall slower as pumping continues. Measurements of water levels must be collected very frequently at the start of the test, and become less frequent as time passes. A commonly used schedule of timings for readings is given in BS ISO 14686:2003. This is summarized as follows:

- Readings to be taken immediately before the discharge flow rate is started, changed, or stopped
- Readings to be taken every minute for the first 10 min of the test phase (if practicable, readings should be taken every 30 s during this period)
- Readings to be taken every 2 min thereafter up to an elapsed time of 20 min
- Readings to be taken every 5 min thereafter up to an elapsed time of 60 min
- Readings to be taken every 10 min thereafter up to an elapsed time of 100 min
- Readings to be taken every 20 min thereafter up to an elapsed time of 300 min
- Readings to be taken every 50 min thereafter up to an elapsed time of 1000 min
- Readings to be taken every 100 min thereafter up to an elapsed time of 3000 min
- Readings to be taken every 200 min thereafter, unless other influences warrant more frequent measurement

These timings are for guidance only and should be applied with judgment. The key point is to ensure that sufficient readings are taken to allow the rate of change of water levels to be clearly identified. If water levels are only changing slowly, it may be possible to increase the intervals between readings without reducing the quality of the data. This would avoid generating unnecessary readings and can reduce the work involved in data analysis.

Water levels can be measured by manual dipping with a dip meter (see Section 16.3). However, at the start of the test, readings need to be taken very frequently. If more than a few observation wells are to be monitored manually, several people may be needed to take the frequent readings at the start of the test, creating potential problems of coordination between personnel. There is also the problem of actually gathering enough capable people together in the first place. This can lead to poor recording of the precise timing of measurements taken during the first few minutes of pumping, leading to difficulties in subsequent analysis. If manually taking water level readings with a limited number of observers, it is best to concentrate initial monitoring on points nearest the test well. As time passes and the period between readings increases, more observation wells can be included in the monitoring.

The use of electronic datalogging equipment, linked to pressure transducers in the observation wells (see Section 16.6), may help overcome such staffing problems. Once programmed to take readings at the appropriate interval, such equipment not only reduces the personnel requirements but will also produce the test measurements in electronic form, allowing rapid analysis using spreadsheet programs.

If a test is carried out in an aquifer with significant tidal response, the monitoring intervals later in the test (after several days of pumping) must be selected with care. Standard tables of monitoring intervals (such as in BS ISO 14686:2003) allow monitoring frequencies of one reading every few hours. If such large time gaps between readings are permitted, data will be very difficult to interpret because the tidal responses will not be accurately recorded. For tidal pumping tests, the monitoring intervals for the first few hours of the test should be as per published guidelines, but the remainder of the test should be monitored at 15–30 min intervals to ensure that tidal fluctuations are fully resolved. This requirement means that a datalogging system is almost essential for pumping tests on tidal sites.

The pump discharge can be measured using a tank or gauge box fitted with a V-notch or rectangular weir (see Section 16.4) or by installing an integrating flowmeter into the discharge pipeline. Even if flowmeters are used in preference to weir tanks, it is good practice to include a settlement tank in the discharge line so that it can be visually checked for suspended solids in the discharge water. In general, once adjusted at the start of the test phase, the flow rate should not vary significantly. Immediately following commencement of pumping, flow rate does not need to be monitored as frequently as water levels for the first hour or so of pumping. The discharge outlet should be well away from the test area so that return seepage does not affect the drawdown levels.

It is normal to take water samples from the pumped discharge during the test. Obtaining a sample is relatively straightforward if it is possible to fill a sample bottle directly at the discharge tank. However, when taking groundwater samples from the discharge flow, the following factors should be considered:

1. Attempt to minimize the exposure of the sample to the atmosphere. Try and obtain it directly at the point where the pump discharges into the tank. Totally fill the bottle and try and avoid leaving any air inside when it is sealed. If the pump discharge is "cascading" before the sampling point, the water will become aerated and oxidation may occur. The discharge arrangements should be arranged so the sample can be obtained before aeration occurs.
2. Samples may degrade between sampling and testing. The samples should be tested as soon as possible after they are taken and, ideally, should be refrigerated in the meantime. The bottles used for sampling should be clean with a good seal. However, the sample may degrade while in the bottle (for example, by trace metals oxidizing and precipitating out of solution). Specialists may be able to advise on the addition of suitable preservatives to prevent this from occurring. The choice of sample bottle (glass or plastic) should also be discussed with the laboratory personnel, because some test results can be influenced by the material of the sample bottle.

The wellhead chemistry (see Section 3.9) can be determined using probes or sensors immersed in the flowing discharge water. These can be portable meters, recorded manually, or may be linked to datalogging systems.

A3.3 EXECUTION OF PUMPING TESTS

Before any pumping is carried out, water levels in the observation wells must be recorded regularly over a period of days. This is to try and determine whether any natural (or artificially induced) variations in water level are occurring. There are no generic guidelines on the duration and frequency of background monitoring. On sites where it is anticipated that variations will be small (e.g., remote inland sites), monitoring three times daily for 3–5 days is the minimum acceptable. On coastal sites with a significant tidal response, it has been necessary to use dataloggers to record water levels every 15 min, 24 h per day for up to 30 days. If in doubt, it is best to do more than the minimum monitoring. The time to find out about background variations is before test pumping, not during the test itself.

The equipment test should be used to determine the most suitable pump discharge rate for subsequent phases of the test, so that at the end of pumping, the water in the borehole is not drawn down to the pump intake. It should also be used to ensure that flow measuring devices function and that discharge pipework is not blocked and does not leak—to check, in general, whether the test is ready to proceed. Ideally, these preparations should be tried out at least a day before test pumping is commenced so that the test well and observation wells can be left to ensure that the groundwater level is realistically reestablished.

The following general information should be recorded before testing:

1. Elevation of ground surface at the test well and at each observation well
2. Elevation of reference datum for water levels at each well (the top of the well casing is often used as a datum)
3. Depth of the well screen in the test well and the depth of response zones in all observation wells
4. Distances from the center of the test well to all observation wells

A step–drawdown test (if carried out) has the aim of investigating the performance of the well at increasing flow rates (see Clark 1977). The well is pumped in a number of steps (ideally four, or an absolute minimum of three); the flow rate in each step is constant, with the rate for each step greater than the last. Increments for flow rate should be roughly equal. For example, a four-step test might be designed as

- Step 1: one quarter of maximum well yield.
- Step 2: one half of maximum well yield.
- Step 3: three quarters of maximum well yield.
- Step 4: maximum well yield (estimated from the equipment test).

Step–drawdown pumping test results can be analyzed to provide information about well efficiency. This is a function of friction head losses through the filter pack and the well screen. It can also be affected by the techniques used to bore and develop the test well.

When starting any pumping (step–drawdown or constant rate) or recovery phase, it is important that the monitoring team has been adequately briefed and is ready. Checks like ensuring all the dip meters work and that all observers have a pen or pencil and paper to record readings are obvious, but can be very embarrassing if overlooked! If datalogging equipment is being used, it should be checked in advance for battery function and for accuracy of clock setting. Above all, one person should be responsible for deciding when to start pumping, and that person should resist being pressured to start before everyone is ready. It is not necessary to start a test on the hour of local time, but it makes recording and analysis easier if the test starts on, for example, a multiple of 10 min past the hour.

The time when the pump is started should be recorded. The control valve of the test pump should be adjusted to achieve the desired flow rate as quickly as possible after the start of pumping (or the step in a step–drawdown test). Once the flow rate has been set, the valve should not be further adjusted as this will affect the drawdown and complicate analysis of results. At the start of the test phase, when readings are being taken frequently, it is important that readings in the test well and observation wells are taken at nearly the same instant as possible. If all the wells are close together, one of the observers may be able to make a visible or audible signal to the others that it is time for a reading. Otherwise, each observer will need to have a clock of some sort, and the clocks will all need to be synchronized. If there are few observers and many wells, it is best to abandon a rigid schedule of monitoring and just take readings as rapidly as possible, but record the precise time each reading was taken.

Monitoring during the test phases normally consists of

1. Water levels in the test well and observation well, with the time of each reading recorded. Readings are normally taken at specified intervals (very frequently at the start of the test, less frequently as pumping proceeds). Guidance is given in BS ISO 14686:2003.
2. Pump discharge flow rate, recorded at the same time as the water levels (during the first hour or so of pumping, it is acceptable to record the flow rate less frequently than water levels, especially if the number of observers is limited).
3. Clarity of discharge water (as a check on any sand or silt in the discharge water).
4. Discharge water temperature and chemistry (by taking samples for testing or using portable meters). It is normal to take a water sample during the constant rate phase immediately after pumping commences, again after a few hours, and just before pumping ceases.

Upon cessation of pumping, monitoring of water levels in piezometers should be continued until full recovery of water level is approached. During the initial recovery period, readings should be at frequent intervals, at similar intervals to the start of pumping. As the rate of water rise slows down, the time intervals between readings may be extended progressively.

A3.4 PLOTTING OF TEST RESULTS

All water level and flow rate data gathered during the test should be plotted in graphical form while the test is proceeding. Even if this is done on-site in rough form on a graph pad, it will help identify any anomalies or inconsistencies. These may be due to occasional human error. If readings are plotted on-site as the test is in progress and an anomaly shows up, a further check reading can be made immediately. If the second reading confirms the earlier reading, it is possible that some fault has developed with the dip meter, datalogger transducer, or observation well and so immediate remedial measures can be implemented.

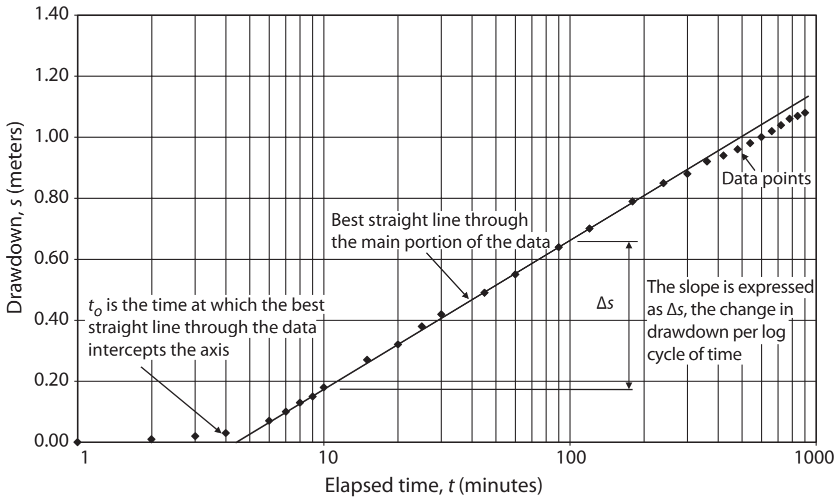

Figure A3.2 Cooper–Jacob straight-line method of plotting data.

Plotting of the test results also allows the aquifer drawdown response to be observed almost in "real time." This can be a useful guide when deciding whether a pumping phase needs to be extended, or whether pumping can be stopped early.

The most useful method of plotting data is to use the Cooper–Jacob straight line method described in Section 6.7. Drawdown is plotted on the vertical axis (linear scale) against elapsed time on the horizontal logarithmic scale (Figure A3.2). In a confined aquifer the first few readings curve upward and, in theory, then form a straight line. Obviously, there will be times when the data will not conform precisely to the theory, but this form of presenting the data is nevertheless very useful for viewing trends in the drawdown data.

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Appendix 4: Design examples

Four design examples, based on the methods in Chapter 7, are presented here. For ease of reference, the relevant equation and figure numbers from the main text are also given.

A4.1 DESIGN EXAMPLE 1

This example is an application of the equivalent well method to analyze a system of fully penetrating deep wells used to lower the piezometric level in a confined aquifer beneath a rectangular excavation. A sensitivity analysis is carried out to assess the effect on the calculated flow rate of various possible values of permeability.

A4.1.1 Conceptual model

A rectangular excavation is to be made to a depth of 9 m. The details of the conceptual model can be summarized as shown in Figure A4.1.

- Excavation dimensions are 35 × 15 m in plan, 9-m depth at the deepest part of the excavation. The excavation is to have vertical sides supported by sheet piles. Dewatering is required for 6 months.
- A confined aquifer, consisting of a medium sand, extends from 10-m depth to 19-m depth. The confining layer above the aquifer is a stiff clay. Maximum piezometric level in the aquifer is 1 m below ground level.
- No pumping test was carried out during site investigation, but particle-size distributions (PSD) and falling head test data can be used to estimate permeability.
- No recharge boundaries are believed to exist. Flow is likely to be radial to the array of wells and can be idealized as a distant circular source.
- No compressible strata are believed to exist. Groundwater lowering–related settlement is not anticipated to be a problem.

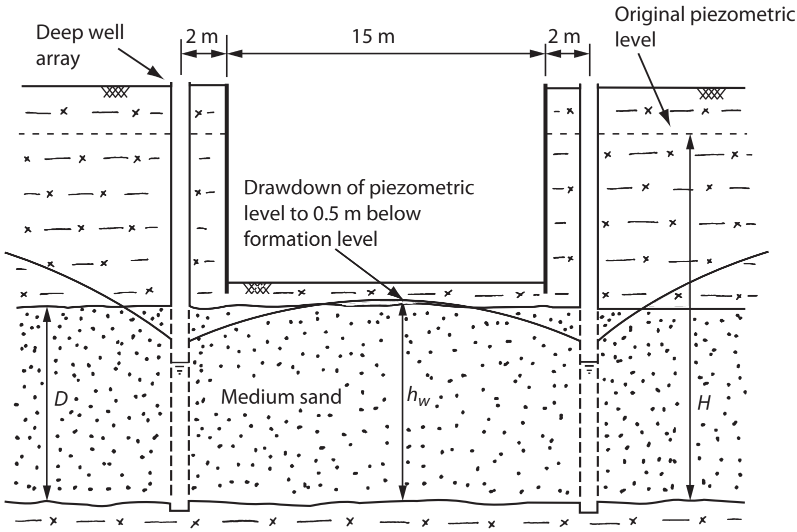

Figure A4.1 Conceptual model for design example 1.

A4.1.2 Selection of method

The presence of a confined aquifer at shallow depth beneath the excavation would result in a risk of base heave if the piezometric level is not lowered. The conservative case is to lower the piezometric level to 9.5 m below ground level (that is, 0.5 m below formation level). This requires a drawdown of 8.5 m below the original groundwater level.

Without assessing the available permeability data in detail, typical values of permeability given in Table 3.1 suggest the permeability k of a medium sand would be in the range 1×10^{-4} to 5×10^{-4} m/s.

Inspection of Figure 5.6 suggests that either deep wells or two-stage wellpoints would be suitable for this combination of drawdown and permeability. In this case, the deep well method will be used, because the contractor wishes to excavate to full depth in one operation, and two-stage wellpointing requires a pause in excavation while the second stage is installed.

A system of relief wells (Section 11.5) might have been considered. However, the anticipated permeability suggests that the flow rate from the relief wells would have been too great to be handled by sump pumping without interfering with construction operations.

A4.1.3 Estimation of steady-state discharge flow rate

The wells are to be installed in a regular pattern around the excavation. This geometry is amenable to solution as an equivalent well. If it is assumed that the wells will be fully penetrating, the total flow rate Q can be estimated from Equation 7.3:

$$Q = \frac{2\pi k D\left(H - h_{\mathrm{w}}\right)}{\ln\left[R_0/r_{\mathrm{e}}\right]} \tag{7.3}$$

where

- k is the aquifer permeability. As described earlier, Table 3.1 suggests a likely permeability in the range 1×10^{-4} to 5×10^{-4} m/s. The site investigation data include PSD data showing a 10% particle size (D_{10}) of 0.1–0.3 mm. The sand is relatively uniform, and Hazen's rule (Equation 6.1) can be used to estimate permeability as 1×10^{-4} to 9×10^{-4} m/s. These permeability estimates are broadly consistent with Table 3.1. The larger end of the range is rather greater than might be expected in a medium sand; the sample may have been affected by "loss of fines" or may represent a coarser layer within the aquifer. Falling head tests in boreholes gave permeabilities in the range 1×10^{-8} to 1×10^{-6} m/s. Comparison with the soil description suggests that these results are unrepresentatively low and are likely to have been affected by silting up during the tests. The falling head test results are not used in subsequent assessments of permeability. It is probably reasonable to assume that design values of permeability are between 1×10^{-4} and 9×10^{-4} m/s and to carry out sensitivity analyses when estimating flow rate.
- D is the aquifer thickness: $D = 19 - 10 = 9$ m.
- $(H - h_{\mathrm{w}})$ is the drawdown: $(H - h_{\mathrm{w}}) = 9.5 - 1 = 8.5$ m.
- r_{e} is the equivalent radius of the array of wells. Assuming that the wells are located 2 m outside the edge of the sheet piles, the overall dimensions of the system will be 39 × 19 m. r_{e} can be estimated from either Equation 7.1

$$r_{\mathrm{e}} = \frac{\left(a+b\right)}{\pi} = \frac{\left(39+19\right)}{\pi} = 18.5 \text{ m} \tag{7.1}$$

or Equation 7.2

$$r_{\mathrm{e}} = \sqrt{\frac{ab}{\pi}} = \sqrt{\frac{39 \times 19}{\pi}} = 15.4 \text{ m} \tag{7.2}$$

Because r_{e} appears in a log term in Equation 7.3, these two values will produce very similar estimates of flow rate; therefore, a value of $r_{\mathrm{e}} = 18.5$ m will be used in subsequent calculations.

- R_0 is the radius of influence. In the absence of pumping test data, R_0 can be estimated for equivalent wells from Equation 7.14 (shown here for R_0, r_e, and $(H - h_w)$ in meters and k in meters per second)

$$R_o = r_e + 3000\left(H - h_w\right)\sqrt{k} \tag{7.14}$$

The total flow rate Q can then be estimated from Equation 7.3 by a sensitivity analysis, within the selected range of permeability. Using the units quoted above, Q will be calculated in cubic meters per second; when commenting on results, Q is normally quoted in liters per second (1 m^3/s = 1,000 L/s) to make the numbers easier to read and interpret.

k (m/s)	*r_e (m)*	*R_0 (m)*	*Q (m^3/s)*	*Q (L/s)*
1×10^{-4}	18.5	274	0.018	18
2×10^{-4}	18.5	379	0.032	32
5×10^{-4}	18.5	589	0.069	69
9×10^{-4}	18.5	784	0.12	120

A4.1.4 Estimation of number of wells

For soils of permeability greater than 1×10^{-4}, q, the maximum yield of a well, can be estimated from Equations 7.30 and 7.31, combined into the following equation (or alternatively, taken from Figure 7.19):

$$q = \frac{2\pi r l_w \sqrt{k}}{15}$$

where l_w is the wetted screen length of wells and r is the radius of the well borehole. For each case of the sensitivity analysis, the total wetted length (all wells in combination) can be estimated.

The number of wells and the corresponding well yield can then be estimated, once certain assumptions have been made about the dimensions of the well. In this case, the diameter of the well borehole (not the diameter of the well screen) is taken to be 0.305 m. The wetted depth per well must also be assumed. The wells fully penetrate the aquifer and have a total screen length of 9 m. However, drawdown within the excavation is to 9.5 m below ground level, compared with the top of the aquifer at 10-m depth. The drawdown in the wells will be greater than the drawdown in the general excavation area; therefore, the wetted length per well will be less than the aquifer thickness of 9 m. In this case, the wetted length per well will be assumed to be 6 m (two-thirds of the aquifer thickness).

The number of wells is the total wetted screen length divided by the wetted screen length per well, with the answer rounded up to the next whole number. The nominal well spacing (assuming that the wells are evenly spaced) is then determined from the plan dimensions of the well system (39 × 19 m). The calculations are shown below.

k (m/s)	*Q (m³/s)*	*Total l_w for all wells (m)*	*No. well and yield (L/s)*	*Nominal well spacing (m)*
1×10^{-4}	0.018	28	5 no. at 3.6 L/s	23
2×10^{-4}	0.032	36	6 no. at 5.3 L/s	19
5×10^{-4}	0.069	48	8 no. at 8.6 L/s	15
9×10^{-4}	0.12	63	11 no. at 10.9 L/s	11

A4.1.5 Empirical checks

It is vital that the basic design is checked against experience or the "normal" range of dewatering systems.

In Section 7.8, it is stated that most deep well systems have a well spacing of between 10 and 60 m. The design fits within these limits, although if the permeability is at the higher end of the analyzed range, it can be seen that the well spacing is at the lower edge of the normal range—this implies that, if permeability is actually rather greater than the analyzed range, then deep wells may not be the most appropriate method.

Another rule of thumb is that, where the aquifer depth allows, dewatering wells should generally penetrate to at least one-and-a-half to two times the depth of the excavation. In this case, the wells are 19 m deep, compared with an excavation depth of 9 m; therefore, this condition is satisfied.

In Table 10.1, the minimum diameters of well bore for a given yield are listed. This table indicates that wells drilled at 305-mm diameter can accommodate pumps of up to 15–20 L/s capacity, within the range of the anticipated yield. These empirical checks confirm, in the gross sense, the validity of the concept of the design.

A4.1.6 Final design

Design calculations have produced a range of flow rates and well yields and spacings. Ultimately, a decision has to be made based in engineering judgment. Any records of local experience may help in forming an opinion. In this case, the nominal system of six or eight wells would seem to be the most appropriate options. If the construction program is tight and cannot cope with delays, the larger system would be prudent. However, in this case, it is assumed that there is time in the program to install a few additional wells (if crude testing of the first wells shows that flow rates are higher than the design value). Accordingly, the nominal six-well system is appropriate.

If the aquifer was unconfined, the calculated steady-state flow rates would need to be increased to allow for additional water from storage release in the early stages of pumping. However, the aquifer is initially confined and, after drawdown, will only become unconfined local to the wells. Confined aquifers have very small storage coefficients; therefore, the water released from storage will not be significant and is not normally allowed for in design.

It is normally prudent to increase the number of wells by approximately 20% to provide some allowance for individual wells being temporarily out of service. In this case, this increases the number of wells from six to eight. At the design yield of 5.3 L/s, this gives a total design capacity of approximately 42 L/s. In practice, the system capacity will be slightly larger, because submersible pumps are manufactured with discrete capacities. The pump chosen will normally have a slightly greater capacity than the design yield. These factors should all help provide a robust design, capable of coping with modest changes in the predicted flow rate.

Because the excavation is fairly narrow, it is likely that a well system of adequate capacity will achieve drawdown everywhere within the excavation area. In many cases, no formal calculation is made of the drawdown distribution within the excavation. If necessary, the methods given in Section 7.9 can be used to obtain *approximate* estimates of drawdown within the excavation.

A4.2 DESIGN EXAMPLE 1A

This example shows an alternative method for the estimation of steady-state flow rate from a system of deep wells in a confined aquifer. The well array is still modeled as an equivalent well, but the flow rate is determined using published “shape factors.” The case analyzed is exactly the same as in design example 1.

A4.2.1 Conceptual model

As design example 1.

A4.2.2 Selection of method

As design example 1.

A4.2.3 Estimation of steady-state discharge flow rate

Flow to rectangular arrays of wells in confined aquifers can be determined by the shape factor method of Equation 7.12.

$$Q = kD(H - h_w)G \tag{7.12}$$

where:

- k is the aquifer permeability. See design example 1 for discussion of selected values of permeability between 1×10^{-4} and 9×10^{-4} m/s.
- D is the aquifer thickness: $D = 19 - 10 = 9$ m.
- $(H - h_w)$ is the drawdown: $(H - h_w) = 9.5 - 1 = 8.5$ m.
- G is a geometry shape factor, obtained from Figure 7.11. To determine the appropriate value of G for a given case, it is necessary to evaluate the following parameters:

Array aspect ratio a/b: if the array of wells is of plan dimensions $a \times b$, the aspect ratio a/b will affect the flow rate. In this case, a/b = 39/19 = 2.1.

L_0/a: the ratio of distance of influence L_0 to the long dimension of the well array a. In the absence of pumping test data, L_0 can be estimated from either Equation 7.13 or Equation 7.15. In this case, the excavation is not very long and narrow, and radial flow is likely to be the dominant flow regime. Therefore, Equation 7.13 is most appropriate.

$$L_0 = 3000\left(H - h_w\right)\sqrt{k} \tag{7.13}$$

Note that Equation 7.14 (which includes the radius r_e of the equivalent well) should not be used with this method because Figure 7.10 is based on distances of influence from the edge of the well array, not the center. If the well array had been long and narrow (i.e., if a had been much greater than b), plane flow would predominate and L_0 should have been estimated from Equation 7.15.

The flow rate is then calculated from Equation 7.12, using values of G from Figure 7.11. The permeability sensitivity analysis of design example 1 has been repeated here.

k (m/s)	*L_0 (m)*	*L_0/a*	*a/b*	*G*	*Q (m^3/s)*	*Q (L/s)*
1×10^{-4}	255	6.5	2.1	2.4	0.018	18
2×10^{-4}	361	9.2	2.1	2.1	0.032	32
5×10^{-4}	570	14.6	2.1	1.9	0.073	73
9×10^{-4}	765	19.6	2.1	1.7	0.12	120

It is unrealistic to expect that the flow rates calculated by two different methods would be precisely the same but, in this case, there is good agreement between the methods used here and in design example 1.

When rounded to two significant figures (as above), the greatest difference between methods is only 3 L/s.

The remainder of the design process is carried out in the same way as for design example 1.

A4.3 DESIGN EXAMPLE 2

This example describes the design of a partially penetrating wellpoint system for trench works in an unconfined aquifer. The line of wellpoints is analyzed as an equivalent slot under plane flow conditions, and the contribution from radial flow to the ends of the slot is assessed. A sensitivity analysis is carried out to assess the effect of varying the depth of the aquifer on the calculated flow rate.

A4.3.1 Conceptual model

A narrow trench excavation is required to allow the laying of a shallow pipeline which extends for several hundred meters. The formation level for the trench is 3 m below ground level. The details of the conceptual model can be summarized as shown in Figure A4.2:

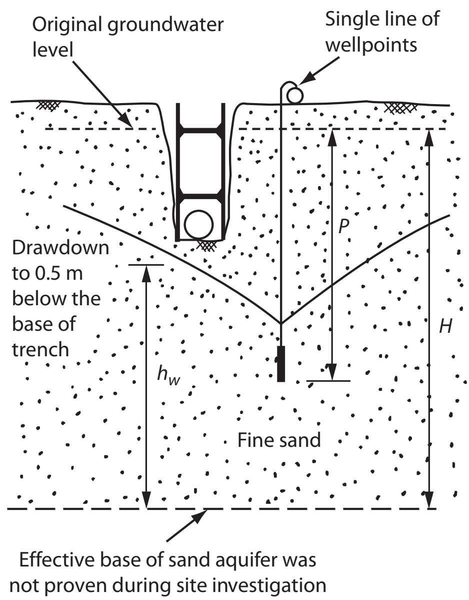

Figure A4.2 Conceptual model for design example 2.

- The excavation is 1 m wide and 3 m in depth. The excavation has near-vertical sides, with the pipelaying operatives working in the trench, protected by a “drag box”—a temporary trench support system. The trench is several hundred meters long, but it is anticipated that no more than 20–30 m will be open at any one time.
- An unconfined aquifer is present beneath the site. The aquifer is composed of uniform fine sand, and groundwater level is generally 0.5 m below ground level.
- Back-analysis of previous groundwater lowering nearby indicates that the sand is of approximate permeability (1×10^{-4} m/s). Boreholes at the current site indicate that ground conditions are very similar to the nearby site, and analysis of PSD results from the current site (using Hazen’s rule; Equation 6.1) confirms this permeability.
- The base of the aquifer was not determined during the investigation. The deepest borehole penetrated to 12 m depth and did not encounter any underlying impermeable stratum. Published geological maps of the area indicate that the stratum of fine sand may extend to 20 m depth or more.
- No recharge boundaries are believed to exist. Flow is likely to be plane to the line of wellpoints and can be idealized as a distant source.
- No compressible strata are believed to exist. Groundwater lowering–related settlement is not anticipated to be a problem.

A4.3.2 Selection of method

The required drawdown (to 0.5 m below formation level) is 3 m (from 0.5 to 3.5 m depth), and the anticipated permeability is 1×10^{-4} m/s. Figure 5.6 suggests single-stage wellpointing, which is a method commonly used for dewatering of pipeline trenches.

Although the trench is very long, only 20–30 m will be open or being worked on at any time; it will take around a week to excavate, lay, and backfill this length of trench. In such circumstances, it is not common practice to dewater the entire length of the trench. Typically, pumping is maintained on a line of wellpoints of length equal to three or four times the weekly rate of advance (see Section 9.8). The wellpoints and pumping equipment are progressed forward to keep pace with the pipelaying. In this design, it is assumed that wellpoints are pumped alongside 100 m of the trench at any one time.

A4.3.3 Estimation of steady-state discharge flow rate

For trench excavations, wellpoints are installed in closely spaced lines parallel to the trench. This geometry is amenable to solution as an equivalent slot—although, as will be described later, it may be necessary to consider radial flow to the end of a line of wellpoints. For a trench depth of 3 m, wellpoints would normally be installed to the standard depth of 6 m. Although the depth of the aquifer is not known, the wellpoints (and hence, the equivalent slot) will be partially penetrating. The total flow rate, Q, for plane flow to a slot in an unconfined aquifer can be estimated from Equation 7.11.

$$Q = \left[0.73 + 0.23\left(P/H\right)\right]\frac{kx\left(H^2 - h_w^2\right)}{L_0} \tag{7.11}$$

where:

- k is the aquifer permeability: k is taken as 1×10^{-4} m/s.
- x is the length of the slot: x is taken as 100 m, the length of the line of wellpoints.
- H is the depth from the original water table to the base of the aquifer and h_w is the depth from the lowered groundwater level (in the equivalent slot) to the base of the aquifer. Note that the drawdown of concern is in (or beneath) the trench itself, rather than at the wellpoints. In reality, the wellpoints will be so close to the trench that there will be little difference in drawdown between the trench and the wellpoints. In calculations, h_w is taken as the head beneath the trench (see Figure A4.2).
- P is the penetration of the slot below the original water table: $P = (6 - 0.5) = 5.5$ m.
- L_0 is the distance of influence. In the absence of pumping test data, L_0 can be estimated for equivalent slots from Equation 7.15 (shown here for L_0, H, and h_w in meters and k in meters per second).

$$L_0 = 1750\left(H - h_w\right)\sqrt{k} \tag{7.15}$$

for $k = 1 \times 10^{-4}$ m/s and $(H - h_w) = 3$ m (drawdown from 0.5 to 3.5 m), L_0 is estimated to be 53 m.

If the depth of the aquifer was known, the total flow rate, Q, could then be estimated from Equation 7.11. The problem of the unknown aquifer depth can be overcome by carrying out a sensitivity analysis—and of course, it is known that the base of the aquifer is more than 12 m below ground level. Using the units quoted above, Q will be calculated in cubic meters per second; when commenting on results, Q is normally quoted in liters per second (1 m^3/s = 1,000 L/s) to make the numbers easier to read and interpret.

Depth to base of aquifer (m)	*H (m)*	*h_w (m)*	*Q (m^3/s)*	*Q (L/s)*
12	11.5	8.5	9.6×10^{-3}	9.6
14	13.5	10.5	1.1×10^{-2}	11
16	15.5	12.5	1.3×10^{-2}	13
18	17.5	14.5	1.4×10^{-2}	14
20	19.5	16.5	1.6×10^{-2}	16

This shows that, in this case, assuming a deeper base to the aquifer increases the flow by almost 70% but, because the initial flow rate was modest, the actual increase in predicted flow is only around 6.5 L/s. This small increase in flow will not significantly affect the design of the system. The effect of a deeper base to the aquifer would have been more problematic if the aquifer permeability was higher because the initial flow rate would have been larger, and a 70% increase would result in a much greater increase in flow.

The above calculations assume plane flow to the sides of the slot, but because the line of wellpoints is of finite length (100 m), there will be some contribution from radial flow to the ends (see Figure 7.4b). The total flow rate to the ends of the slot is the same as the flow rate to a well of radius, r_e, equal to half the width of the slot. For a partially penetrating well in an unconfined aquifer, the flow rate from such a well can be estimated from Equations 7.5 and 7.7 and combined as

$$Q = \frac{B\pi k\left(H^2 - h_w^2\right)}{\ln\left[L_0/r_e\right]}$$

Where all the terms are as defined previously, apart from B, which is a partial penetration factor for radial flow to wells, determined from Figure 7.10b. This equation has been used to calculate the contribution from radial flow to the ends of the slots—the flow rate is theoretically split 50–50 at either end of the slot, but it is the total flow that is relevant now. In calculations, it has been assumed that the slot is a single line of wellpoints of width 0.2 m (a typical width of the jetted hole formed by a placing tube); therefore, r_e was taken as 0.1 m. In this case, the contribution from radial flow to the ends of the slot is small (between 8% and 15% of the flow to the sides). Obviously, if the slot had not been so long, the percentage contribution from the ends would have been greater.

Depth to base of aquifer (m)	*Q plane flow to both sides only (L/s)*	*Q radial flow to ends (L/s)*	*Q total flow: plane plus radial (L/s)*
12	9.6	1.4	11
14	11	1.4	12
16	13	1.4	14
18	14	1.4	15
20	16	1.4	17

If it was anticipated that the wellpoint system would have been installed as a double-sided system, consisting of two parallel lines of wellpoints, the flow to the ends would have been greater. If the two lines of wellpoints were, say, 5 m apart, the radial flow calculation would have used (r_e = 2.5 m), and the total flow to both ends of the slot would have been between 25% and 45% of the flow to the sides. This highlights that it is more important to consider the flow to the ends of a double-sided wellpoint system than it is for a single-sided system.

A4.3.4 Determination of wellpoint spacing and pump capacity

It is likely that a single-sided wellpoint system (consisting of a line of wellpoints along one side of the trench only) will be appropriate for this excavation. This case satisfies the conditions favorable to single-sided wellpointing set out in Section 9.8.

1. A narrow trench.
2. Effectively homogeneous, isotropic permeable soil conditions that persist to an adequate depth below formation level (see Figure 9.1).
3. Trench formation level is not more than approximately 5 m below standing groundwater level.

Table 9.1 gives typical wellpoint spacing in sands as 1–2 m. The length of the proposed wellpoint system is 100 m. Assuming an initial spacing of 2 m, fifty wellpoints will be pumped at any one time. If wellpoints of standard screen length (0.7 m) are installed by placing tube, Figure 7.20 suggests that for a permeability of 1×10^{-4} m/s, each wellpoint would have a capacity of 0.26 L/s. This gives a maximum yield of 13 L/s for a fifty-wellpoint system. This is lower than the predicted flow rate if the aquifer is 20 m deep; therefore, it would be prudent to install the wellpoints at 1.5 m centers, giving a system of sixty-seven wellpoints, with a maximum yield of 17 L/s, which is acceptable for the maximum predicted flow rate. On routine projects, wellpoint spacings tend to be used increments of 0.5 m. If a spacing of 1.5 m was not satisfactory, the next case to be attempted would be 1.0 m spacings. It is rare to consider spacings such as 1.4, 1.3 m, and so on.

If the yield from each wellpoint is acceptable for the steady-state case, it is almost certainly adequate to deal with the additional flow rate from storage release during the initial drawdown period. Because until the steady-state drawdown develops, the wetted screen length of the wellpoints will be much greater, allowing them to yield more water.

The wellpoint pump(s) must also be selected. The choice of pump may be influenced by the equipment available locally, and sometimes much larger pumps than strictly necessary are provided, purely because they are close to hand. Whatever pumps are used, the pump capacity must be adequate for the predicted flow rate, not only the steady-state discharge but also for the additional water released from storage during initial pumping.

A simplistic calculation using Equation 7.29 (and assuming the sand has a storage coefficient S of 0.2) estimates that the design steady-state distance of influence of 53 m will take around half a day to one day to develop. This gives a very crude estimate of the time during which water released from storage will be significant and also roughly correlates with the time to achieve drawdown close to the line of wellpoints. A drawdown period of around one day may seem fairly quick, but pipelaying is a progressive operation, moving forward the entire time. Each wellpoint may only be pumped for a week or so until it is turned off after the trench has passed. New wellpoints are continually being installed and commissioned ahead of pipelaying. A rapid drawdown period is essential to avoid the pipelaying operation moving too fast and advancing ahead of the dewatered area into "wet" ground, where the target drawdown has not yet been achieved. This event should be avoided as it can waste a lot of time and money. This problem does occur from time to time, and when it does, the pipelaying operatives are often forthright (to say the least) in their criticism of the groundwater lowering operation. For "static" or nonprogressive excavations, there is often a little less pressure on achieving very rapid drawdowns.

In this case, it is assumed that double-acting piston pumps are available. In Section 13.2, a 100 mm unit is quoted as having a capacity of up to 18 L/s. This could handle the predicted steady-state flow rate, but may not be able to cope with the higher flows to establish drawdown. A larger 125 mm unit has a capacity of 26 L/s, approximately 50% greater than the maximum predicted steady-state discharge, giving useful spare capacity to deal with water released from storage. If 125 mm pumpsets were not available, two 100-mm units could be provided as an alternative. Both units would be pumped during the initial drawdown period, but later, when the flow rate has reduced, it may be possible to maintain drawdown using one pump only. The other pump would remain connected into the system as a standby. Even if large-capacity pumps are available, the need for standby pumps must be considered; conditions when standby pumps are required are outlined in Section 9.7.

A4.4 DESIGN EXAMPLE 3

This example is an application of cumulative drawdown analysis (theoretical method), to analyze a system of fully penetrating deep wells used to lower the piezometric level in a confined aquifer beneath a rectangular excavation. Aquifer parameters determined from a pumping test are used to allow the estimation of drawdown at specified locations around the excavation area.

A4.4.1 Conceptual model

A rectangular excavation is to be made to a depth of 12 m. The details of the conceptual model can be summarized as shown in Figure A4.3:

- Excavation dimensions are 50 × 50 m in plan at formation level, with the deepest part of the excavation at 12 m depth. The sides of the excavation are battered back at 1 in 1.5, giving overall dimensions of 86 × 86 m at ground level. To try and keep any dewatering wells slightly closer to the deepest part of the excavation, they will be installed on a bench in the excavation batters at a level of 2 m below ground level. The plan dimensions of the well array will be 80 × 80 m. The excavation was to be dewatered for a period of 5 months, and the construction program required that drawdown be achieved within 2 weeks of commencing pumping.
- A confined aquifer, consisting of a sandy gravel, extends from 10 m depth to 26 m depth. The confining layer above the aquifer is a stiff clay. Maximum piezometric level in the aquifer is 5 m below ground level.
- A well pumping test was carried out, pumping at a rate of 15 L/s for 7 days. The discharge during the test was limited by the capacity of the

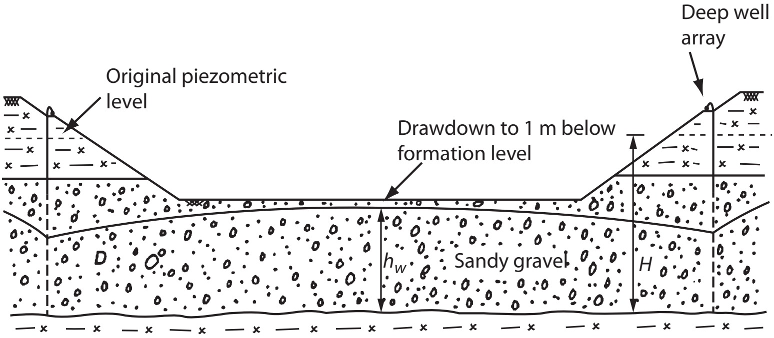

Figure A4.3 Conceptual model for design example 3.

pump; if a larger pump had been available, a greater flow rate would have been possible. Analysis of the pumping test data gave an aquifer permeability, k, of 6×10^{-4} m/s and a storage coefficient, S, of 0.001.
- No compressible strata are believed to exist. Groundwater lowering–related settlement is not anticipated to be a problem.

A4.4.2 Selection of method

The excavation extends through the stiff clay aquiclude and into the upper few meters of the confined aquifer. The piezometric level in the confined aquifer will need to be lowered before excavation to prevent base heave during excavation through the clay and then to provide a workable excavation when the excavation penetrates into the top of the aquifer.

The target drawdown is to lower the groundwater level to 1 m below the formation level. This is 13 m in depth, or a drawdown of 8 m below the original piezometric level.

For a drawdown of 8 m and the design permeability of 6×10^{-4} m/s, inspection of Figure 5.6 suggests that either deep wells or two-stage wellpoints would be suitable for this combination of drawdown and permeability. In this case, the deep well method will be used because the contractor wishes to excavate rapidly to full depth.

A4.4.3 Estimation of steady-state discharge flow rate and estimation of number of wells

The cumulative drawdown method (using the Cooper–Jacob simplification) can be used in confined aquifers. It has also been successfully applied in unconfined aquifers in which the final drawdown is approximately less than 20% of the initial saturated aquifer thickness. In this case, because drawdown is required to 13 m depth compared with the top of the aquifer at 10 m depth, the initially confined aquifer will become unconfined. The aquifer thickness will be reduced by 3 m out of 16 m, or 19%. Therefore, this problem will be analyzed assuming confined behavior throughout.

The cumulative drawdown is calculated using Equation 7.18.

$$(H-h)=\sum_{i=1}^{n}\frac{q_i}{4\pi kD}\left\{-0.5772-\ln\left[\frac{r_i^2 S}{4kDt}\right]\right\} \tag{7.18}$$

where:

- $(H - h)$ is the cumulative drawdown (at the point under consideration) resulting from n wells, each pumped at constant flow rate q_i.
- k is the aquifer permeability: k is taken as 6×10^{-4} m/s.

- S is the aquifer storage coefficient: S is taken as 0.001.
- D is the original aquifer saturated thickness: $D = 26 - 10 = 16$ m.
- t is the time since pumping began. In this case, the target drawdown is required within 14 days. It is always prudent to design to obtain the drawdown a little quicker than planned—this allows for minor problems during commissioning. In design, we will aim to achieve the target drawdown within 10 days. $t = 86{,}400$ s will be used in calculations.
- r_i is the distance from each pumped well to the point where drawdown is being estimated.

Equation 7.18 is valid provided $u = (r^2S)/(4kDt)$ is less than 0.05. In this case, taking r to be 113 m (the distance from the corners of the well array to the center of the excavation) u is less than 0.05 after approximately 2 h; therefore, this method can be used for all t greater than 2 h.

The method requires that the plan layout of the well array be sketched, and the xy-coordinates of each well be determined. The coordinates then allow the radial distances r_i (from each well to the point where drawdown is being checked) to be calculated. An initial guess is made of the number of wells, well spacing, and the resulting x–y coordinates determined. In this case, the initial guess was sixteen wells evenly spaced at 20 m centers (Figure A4.4).

A spreadsheet program is then used to evaluate Equation 7.18 for the cumulative drawdown at selected locations within the excavation. For circular or rectangular excavations with evenly spaced wells, it is normally sufficient to determine the drawdown in the center of the excavation because drawdown everywhere else will be greater. This is the method used here. If the well array is irregular in shape (or if the depth of excavation is not constant) it will be necessary to determine the drawdown in a number of locations, to ensure the target drawdown is achieved at all critical locations.

The results from a spreadsheet calculating the drawdown in the center of the excavation for a 16-well system are shown below. The radial distance, r_i, from each well (at location x_i, y_i) to the location (x_c, y_c) where the drawdown is being determined, is calculated from:

$$r_i = \sqrt{\left(\left[x_i - x_c\right]^2 + \left[y_i - y_c\right]^2\right)}$$

For simplicity, the flow rate q_i from each well has been assumed to be the same, but if it was intended to use pumps of different sizes in certain wells this can easily be incorporated in the calculation. In the spreadsheet different values of q_i were tried until the target drawdown of 8 m is just achieved in the center of the excavation. The total flow rate is simply the sum of all the well flow rates.

Well	*x coordinate (m)*	*y coordinate (m)*	*Well flow rate q_i (m^3/s)*	*Radial distance r_i (m)*	*Drawdown $(H - h)_i$ (m)*
1	0	0	7.0×10^{-3}	56.5	0.50
2	0	20	7.0×10^{-3}	44.7	0.53
3	0	40	7.0×10^{-3}	40.0	0.54
4	0	60	7.0×10^{-3}	44.7	0.53
5	0	80	7.0×10^{-3}	56.6	0.50
6	20	80	7.0×10^{-3}	44.7	0.53
7	40	80	7.0×10^{-3}	40.0	0.54
8	60	80	7.0×10^{-3}	44.7	0.53
9	80	80	7.0×10^{-3}	56.6	0.50
10	80	60	7.0×10^{-3}	44.7	0.53
11	80	40	7.0×10^{-3}	40.0	0.54
12	80	20	7.0×10^{-3}	44.7	0.53
13	80	0	7.0×10^{-3}	56.6	0.50
14	60	0	7.0×10^{-3}	44.7	0.53
15	40	0	7.0×10^{-3}	40.4	0.54
16	20	0	7.0×10^{-3}	44.7	0.53
		Total flow rate	**0.112 m^3/s** (112 L/s)	**Total drawdown**	**8.43 m**

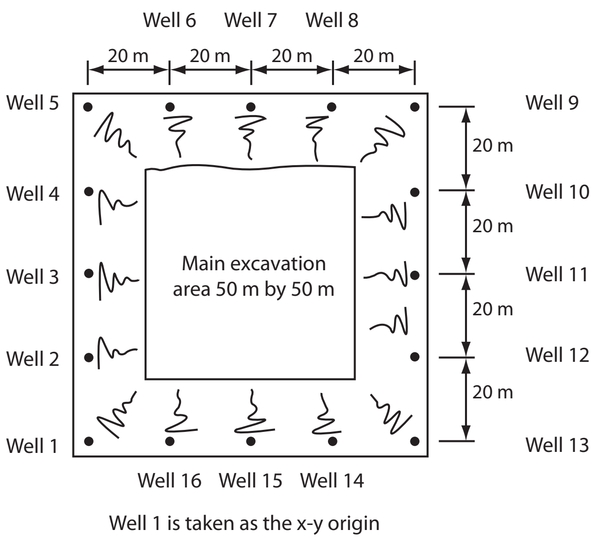

Figure A4.4 Schematic plan of 16-well system.

This calculation indicates that a system of sixteen wells, each discharging 7 L/s (total flow rate 112 L/s) will achieve the target drawdown in the center of the excavation after 10 days. During the pumping test, the test well produced 15 L/s and could have yielded more if a larger pump had been used. It therefore makes sense to repeat the above calculations assuming fewer wells of greater discharge rate. The results of these calculations are summarized below.

No. of wells	*Well spacing (m)*	*Well discharge (L/s)*	*Drawdown in center of excavation (m)*	*Total flow rate (L/s)*
16	20	7.0	8.43	112
13	25	8.5	8.33	110.5
10	32	11.0	8.28	110
8	40	13.5	8.07	108

It is apparent that the target drawdown can be achieved using various combinations of well numbers and yields, but that the total flow rate remains approximately constant.

A4.4.4 Final design and empirical checks

The number and yield or wells chosen for the final design will depend on several factors, including:

1. The need for redundancy in a well system. Any system relying on relatively few wells is vulnerable to one or two wells experiencing damage or pump failure, leading to loss of drawdown and flooding, or instability of the excavation. A system consisting of a greater number of wells will lose proportionately less drawdown if one or two wells are lost.
2. Each well must be able to yield the discharge flow rate q_i assumed in design. Because the pumping test well produced 15 L/s (and could have yielded more), and because all the current designs use q_i of less than 15 L/s, it is likely that all the current designs are feasible. In any case, the theoretical maximum well yield can be estimated from Equations 7.30 and 7.31 (or from Figure 7.19). Assuming a well borehole diameter of 0.305 mm and a wetted screen length of 10–12 m (i.e., a drawdown at the wells of 1–3 m below the excavation target drawdown level), the maximum well yield is estimated at 16–19 L/s. However, in practice, problems can occur if the dewatering wells are not designed, installed, and developed in exactly the same way as the test well—this may cause the production wells to have lower yields than the test wells. Also, in fissured rock aquifers, well yields may

vary significantly. Some wells can have high yields and yet others, poorly connected into fissures, may be almost “dry.” In fissured aquifers, the cumulative drawdown method needs to be applied with care.

In this case it is assumed that, because of the availability of pumps of suitable capacity, the nominal system of ten wells, each discharging 11 L/s each, will be adopted. It is normal practice to apply an empirical superposition factor J; the system capacity is increased by a factor of $1/J$ (see Equation 7.19). This empirical factor allows for interference between wells and also provides some allowance for additional drawdown around the wells and water released from storage when the aquifer becomes unconfined. Where aquifers become unconfined and drawdowns are small (less than 20% of the initial saturated aquifer thickness), the empirical superposition factor J is normally taken as 0.8–0.95. In this case, because the drawdown will reduce the thickness of the aquifer by almost 20%, the maximum superposition factor of 0.8 will be applied; therefore, the system capacity (and hence, the number of wells) will need to be increased by 1/0.8 = 1.25.

The final system design is, therefore, for thirteen wells, each of 11 L/s capacity. Total system capacity is 143 L/s. Table 10.1 indicates that, to accommodate a pump of suitable capacity, a minimum well bore diameter of 300 mm is required. The corresponding well screen and liner diameter is 165 mm.

The design is then verified with some simple empirical checks. The well spacing for a 13-well system is approximately 25 m; this is within the “normal” 10–60 m range quoted in Section 7.8. The wells are intended to fully penetrate the aquifer and are therefore 26 m deep, just over twice the depth of the excavation. Again, this is consistent with guidelines given in Section 7.8. These empirical checks confirm, in a gross sense, the validity of the concept of the design.

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Appendix 5: Estimation of flow rate using V-notch weirs

V-notch weirs are a common method for the estimation of discharge flow rate in the field.

V-notch weirs are a form of thin plate weir, where the area of flow is a notch cut in the shape of a "V" with an internal angle of α (see Figure A5.1), normally installed in a tank. The flow rate over the weir is a function of the head over the weir, the size and shape of the discharge area, and an experimentally determined discharge coefficient.

BS 3680:1981 provides formula and discharge coefficients for V-notch weirs (and the less commonly used rectangular notch weirs). These formulas have been used to produce calibration charts for various standard V-notch angles; these charts are presented at the end of this appendix.

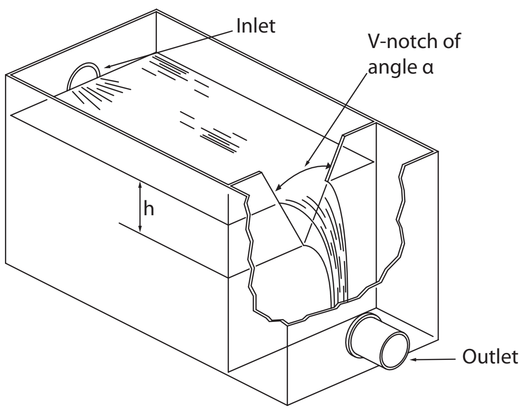

Figure A5.1 V-notch weir for measurement of discharge flow rate. (From Preene, M. et al., Groundwater control—design and practice, Construction Industry Research and Information Association, *CIRIA Report C515*, London, 2000. Reproduced by kind permission of CIRIA: www.ciria.org.)

Knowing the angle of the V-notch, a measurement of the head of water above the base of the notch allows the flow rate to be estimated from the appropriate chart. The position of measurement should be upstream from the weir plate by a distance of approximately 1.1–0.7 m but not near a baffle or in the corner of the tank. The tank should be positioned on a firm stable base, with timber packing used to ensure that the V-notch plate is vertical and that the top of the notch is horizontal.

The calibration charts given below are based on generic formulas and discharge coefficients and are intended for use only to provide field estimates of flow rate from groundwater lowering systems. BS 3680:1981 gives guidance on the use of thin plate weirs for more accurate measurements. Calibration charts are provided for the following notch angles:

Name	*V-notch angle*	*Calibration chart*
90° V-notch	90°	Figure A5.2
1/2 90° V-notch	53° 8′	Figure A5.3
1/4 90° V-notch	28° 4′	Figure A5.4
60° V-notch	60°	Figure A5.5
45° V-notch	45°	Figure A5.6
30° V-notch	30°	Figure A5.7

The angles of 1/2 90° and 1/4 90° V-notch weirs are not 45° and 22.5° as might be expected. These weirs are so called, because they can pass one-half and one-quarter as much flow as a 90° V-notch weir.

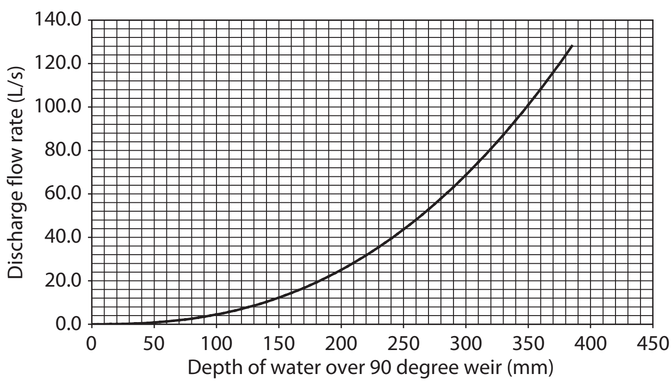

Figure A5.2 Calibration chart for 90° V-notch.

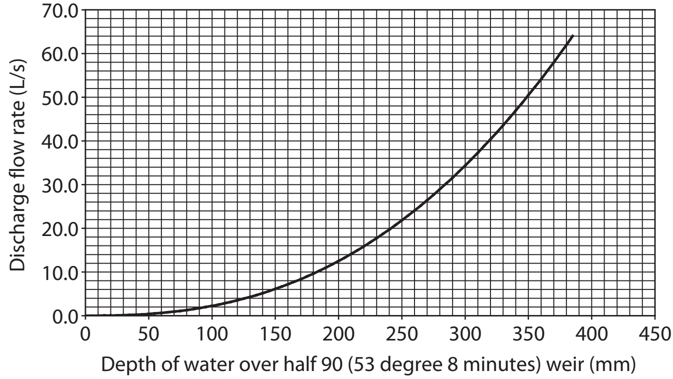

Figure A5.3 Calibration chart for 1/2 90° V-notch.

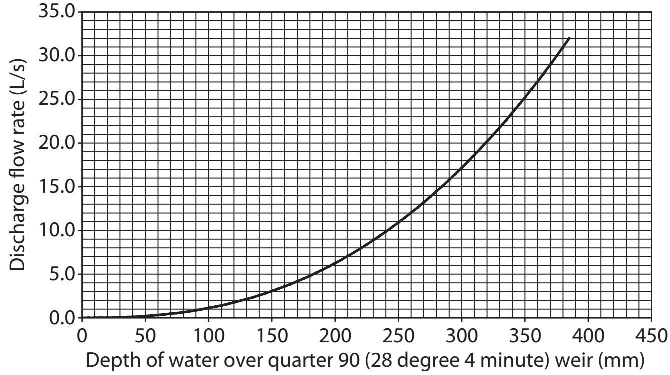

Figure A5.4 Calibration chart for 1/4 90° V-notch.

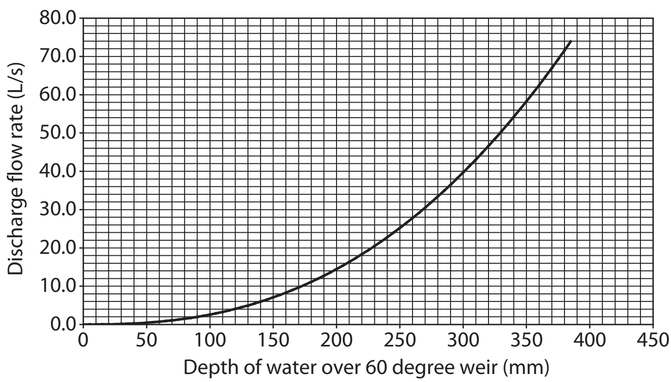

Figure A5.5 Calibration chart for 60° V-notch.

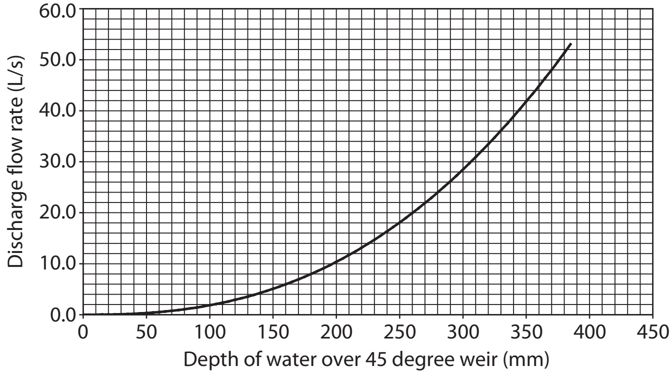

Figure A5.6 Calibration chart for 45° V-notch.

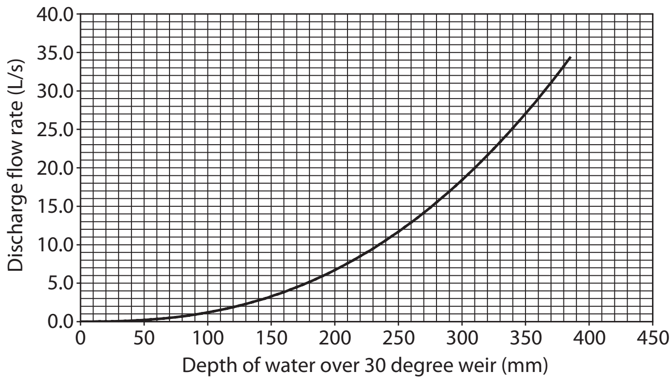

Figure A5.7 Calibration chart for 30° V-notch.

List of notation

A	Area; Cross-sectional area of borehole casing
a	Calibration coefficient; length of well array; length of excavation
B	Calibration coefficient; partial penetration factor for radial flow
b	Calibration coefficient; width of well array; width of excavation
C	Calibration coefficient
C_d	Correction factor for effective stress
c_h	Coefficient of consolidation of soil for vertical compression of soil under horizontal drainage
c_v	Coefficient of consolidation of soil
D	Aquifer thickness; diameter of borehole test section; thickness of soil layer; diameter of tunnel
D_{10}	Sieve aperture through which 10% of a soil sample will pass
D_{15}	Sieve aperture through which 15% of a soil sample will pass
D_{40}	Sieve aperture through which 40% of a soil sample will pass
D_{50}	Sieve aperture through which 50% of a soil sample will pass
D_{60}	Sieve aperture through which 60% of a soil sample will pass
D_{70}	Sieve aperture through which 70% of a soil sample will pass
D_{85}	Sieve aperture through which 85% of a soil sample will pass
d	Penetration of cutoff wall below lowered groundwater level in cofferdam
d_1	Drawdown of groundwater level in cofferdam
dl	Length of flow path
E'_o	Stiffness of soil in one-dimensional compression
EC	Specific conductivity of water
e	Void ratio of soil
F	Factor of safety; shape factor for permeability test in borehole or observation well
f	Angularity factor of soil grains
G	Geometry shape factor for flow to rectangular equivalent wells in confined aquifers
H	Excess head in test section; initial groundwater head; head of groundwater above tunnel axis

H_c	Constant excess head during constant head permeability test
H_o	Excess head in test section at $t = 0$
h	Total hydraulic head; groundwater head; maximum drainage path length for vertical drainage; depth of water over weir
h_w	Groundwater head in pumped well or slot; groundwater head in excavation
dh	Difference in hydraulic head
$(H - h)$	Drawdown
$(H - h_w)$	Drawdown in a pumped well or slot
i	Hydraulic gradient
i_{crit}	Critical hydraulic gradient
i_{max}	Maximum hydraulic gradient at entry to a well; maximum hydraulic exit gradient into an excavation
i_{maxr}	Maximum hydraulic exit gradient into an excavation when a lower permeability layer is present
J	Empirical superposition factor
k	Permeability (also known as coefficient of permeability or hydraulic conductivity)
k_e	Calibration factor between total dissolved solids and specific conductivity
k_h	Permeability in the horizontal direction
k_r	Permeability of the layer of lower permeability
k_v	Permeability in the vertical direction
L	Length of borehole test section
L_o	Distance of influence for plane flow
l	Length of flow path
l_w	Wetted screen length of a well
m_v	Coefficient of volume compressibility of soil
N	Standard penetration test blow count
n	Porosity of a soil or rock; number of pumped wells
P	Depth of penetration into aquifer of partially penetrating well or slot
Q	Flow rate; discharge flow rate from an equivalent well or slot
Q_{fp}	Discharge flow rate from a fully penetrating well
Q_{pp}	Discharge flow rate from a partially penetrating well
q	Flow rate; discharge flow rate from a well; discharge flow rate per unit length of excavation
q_c	Calculated steady-state dewatering flow rate
q_r	Steady-state dewatering flow rate recorded in the field; discharge flow rate per unit length of excavation when a lower permeability layer is present
R	Average degree of consolidation of a soil layer
R_o	Radius of influence for radial flow
r	Distance from the test well to an observation well; radial distance from a well; radius of a well borehole

r_e	Equivalent radius of a well array
r_o	Equivalent radius of a penetrometer
S	Storage coefficient of aquifer; specific surface of soil grains
S_y	Specific yield of unconfined aquifer
s	Drawdown; distance from base of cutoff wall to deeper impermeable layer
s_w	Drawdown at a pumped well or slot
Δs	Change in drawdown per log cycle
T	Transmissivity of aquifer; basic time lag for permeability test in borehole or observation well
T_r	Time factor for radial groundwater flow
T_v	Time factor for vertical drainage
T_{50}	Time factor for 50% dissipation of pore water pressure
TDS	Total dissolved solids
t	Time since pumping began; time since permeability test started; Thickness of lower permeability layer
t_o	Time at which a straight line through the observation well data intercepts the zero drawdown line in a Cooper–Jacob time-drawdown plot
t_{50}	Time for 50% dissipation of pore water pressure
U	Degree of dissipation of pore water pressure; uniformity coefficient
u	Pore water pressure; argument of Theis well function
u_i	Pore water pressure at the start of a permeability test
u_o	Equilibrium pore water pressure
u_t	Pore water pressure at time t
Δu	Reduction in pore water pressure
$W(u)$	Theis well function
x	Linear distance from a slot; length of a pumped slot; length of tunnel; fraction of the total mass of a soil sample
z	Depth below the water table in an unconfined aquifer
α	Internal angle of V-notch in thin plate weir
ϕ'	Angle of shearing resistance of soil
γ_s	Unit weight of soil
γ_w	Unit weight of water
λ	Partial penetration factor for confined slots
ρ_{corr}	Corrected compression of a soil layer
ρ_t	Effective compression of a soil layer at time t
ρ_{total}	Ground settlement
ρ_{ult}	Ultimate compression of a soil layer
σ	Total stress
σ'	Effective stress
μ	Flow reduction factor for seepage into a cofferdam
τ	Shear strength of soil
τ_f	Shear strength of soil at failure

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List of conversion factors

Length

Conversion from metric to imperial			Conversion from imperial to metric		
From	*To*	*Multiply by*	*From*	*To*	*Multiply by*
km	mi	0.622	mi	km	1.609
m	yd	1.094	yd	m	0.914
m	ft	3.281	ft	m	0.305
cm	in	0.394	in	cm	2.54
mm	in	0.394	in	mm	25.4

Note: In the metric system, 1 km = 1000 m; 1 m = 100 cm; 1 m = 1000 mm.

Volume

Conversion from metric to imperial			Conversion from imperial to metric		
From	*To*	*Multiply by*	*From*	*To*	*Multiply by*
m^3	cu yd	1.309	cu yd	m^3	0.764
m^3	cu ft	35.32	cu ft	m^3	0.028
m^3	gal (U.K.)	220	gal (U.K.)	m^3	0.0045
m^3	gal (U.S.)	264	gal (U.S.)	m^3	0.0038
L	gal (U.K.)	0.220	gal (U.K.)	L	4.546
L	gal (U.S.)	0.264	gal (U.S.)	L	3.785
L	pint (U.K.)	1.76	pint (U.K.)	L	0.568
L	pint (U.S.)	2.11	pint (U.S.)	L	0.473

Note: In the metric system, 1 m^3 = 1000 L; 1 m^3 = 10^6 cm^3; 1 cm^3 = 1 mL.

Flow rate

Conversion from metric to imperial			Conversion from imperial to metric		
From	*To*	*Multiply by*	*From*	*To*	*Multiply by*
m^3/day	gal/day (U.K.)	220	gal/day (U.K.)	m^3/day	0.0045
m^3/day	gal/day (U.S.)	264	gal/day (U.S.)	m^3/day	0.0038
m^3/h	gal/h (U.K.)	220	gal/h (U.K.)	m^3/h	0.0045
m^3/h	gal/h (U.S.)	264	gal/h (U.S.)	m^3/h	0.0038
L/s	gal/min (U.K.)	13.2	gal/min (U.K.)	L/s	0.076
L/s	gal/min (U.S.)	15.85	gal/min (U.S.)	L/s	0.063

Note: In the metric system, 1 m^3/s = 1000 L/s; 1 m^3/h = 3.6 L/s; 1 m^3/day = 0.0115 L/s.

Permeability (or hydraulic conductivity or coefficient of permeability)

Conversion from metric to imperial			Conversion from imperial to metric		
From	*To*	*Multiply by*	*From*	*To*	*Multiply by*
m/s	ft/day	2.83×10^5	ft/day	m/s	3.53×10^{-6}
m/day	ft/day	1.04×10^8	ft/day	m/day	0.305
m/year	ft/year	3.281	ft/year	m/year	0.305

Note: In the metric system, 1 m/s = 8.64×10^4 m/day; 1 m/s = 3.16×10^7 m/year.

Pressure and head

Conversion from metric to imperial			Conversion from imperial to metric		
From	*To*	*Multiply by*	*From*	*To*	*Multiply by*
mm Hg	in Hg	0.0394	in Hg	mm Hg	25.4
m H_2O	in Hg	2.896	in Hg	m H_2O	0.345
m H_2O	ft H_2O	3.281	ft H_2O	m H_2O	0.3048
m H_2O	psi	1.42	psi	m H_2O	0.703
kPa	psi	0.1451	psi	kPa	6.884
bar	in Hg	29.551	in Hg	bar	0.0339
bar	ft H_2O	33.48	ft H_2O	bar	0.0299
bar	psi	14.51	psi	bar	0.0689

Note: In the metric system, 1 m H_2O = 9.789 kPa; 1 bar = 100 kPa; 1 bar = 750 mm Hg.

Abbreviations

cm	centimeter
cu	cubic
ft	foot
ft H_2O	foot head of water
gal	gallon
h	hour
in	inch
in Hg	inches of mercury
km	kilometer
kPa	kilopascal
L	liter
m	meter
m H_2O	meter head of water
mL	milliliter
mm	millimeter
mm Hg	millimeters of mercury
min	minute
psi	pounds per square inch
s	second
V	volt
yd	yard

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Glossary

Abstraction The pumping or removal of groundwater (such as from a *well* or *sump*) or surface water (such as from a lake or river).

Air lift A means of pumping water from a well using compressed air to aerate the water so that the air–water mixture rises to the surface. The method is capable of pumping silt and sand with the water and is often used as a method of *well development*.

Air stripping A form of *groundwater treatment* used to remove volatile hydrocarbon compounds from water. The process involves air being blown through a stream or spray of water while it cascades over an assemblage of plastic media, which have very high surface area. The airflow encourages the volatile compounds to escape as vapor, leaving the water less contaminated.

Anisotropic Having one or more physical properties that vary with the direction of measurement—the converse of *isotropic*.

Aquiclude Soil or rock forming a stratum, group of strata, or part of stratum of very low permeability, which acts as an effective barrier to groundwater flow. This term is obsolete in American terminology, where "confining bed" is the equivalent term.

Aquifer Soil or rock forming a stratum, group of strata, or part of stratum from which water in usable quantities can be *abstracted*. By definition, an aquifer will be water bearing (i.e., saturated and permeable).

Aquitard Soil or rock forming a stratum, group of strata, or part of stratum of intermediate to low permeability, which yields only very small groundwater flows or allows groundwater to pass through very slowly.

Artificial ground freezing The process of reducing the temperature of the ground sufficiently to form a very low permeability freezewall of frozen soil. The freezewall can be used as a *cutoff wall* and support structure for belowground works.

Artificial recharge The deliberate reinjection of water (via pits, trenches, or wells) into aquifers or aquitards. Sometimes used as a means of reducing drawdowns in the aquifer outside an excavation to minimize *consolidation settlements*.

Barrier boundary An aquifer boundary that is not a source of water.

Base instability Uplift, heave, or instability of the floor of an excavation as a result of unrelieved or uncontrolled pore water pressures at depth.

Bentonite slurry A suspension of bentonite (a clay mineral consisting of sodium montmorillonite) in water. The slurry is commonly used to support the sides of temporary trench excavations used to construct *diaphragm walls* and *slurry trench walls*.

Biofouling The clogging or deterioration of performance of wells, pumps, and pipework as a result of bacterial growth.

Boil The turbulent jet of soil and water that rises to the surface around a placing tube when a wellpoint or well is installed by *jetting*. Also used to describe an uncontrolled upward seepage into an excavation that may wash *fines* out of the soil, leading to instability.

Bored pile wall *Cutoff walls* formed from circular concrete piles installed in close proximity (in some cases intersecting with each other) to form a line to act as a barrier to groundwater flow and, potentially, to support the perimeter of the excavation. There are two principal configurations of bored pile wall: *secant pile walls* and *contiguous pile walls*.

Borehole Strictly, a borehole is a hole drilled into the ground for any purpose, including site investigation boreholes. In groundwater terminology, a borehole is often taken to mean a relatively small diameter *well*, particularly one used for water supply.

Buoyancy uplift A form of *base instability* that can occur when a permeable stratum exists at a relatively shallow depth beneath an excavation. If the upward force from the groundwater pressures exceeds the weight of the "plug" of soil beneath the excavation, there is a risk that buoyancy forces may cause heave or instability of the base of the excavation.

Capillary saturated zone The zone immediately above the *water table* in an unconfined aquifer where the soil remains *saturated* at negative (i.e., less than atmospheric) pore water pressures. The water in this zone is continuous with the pore water below the water table. The height of the capillary saturated zone is greater in finer grained soils than in coarse-grained soils. The zone rises or falls with any variations in the water table.

Cement-based grout A type of *suspension grout* consisting of a mixture of cement and water, sometimes with additives such as bentonite or pulverized fuel ash.

Cement–bentonite wall A type of *slurry trench wall* where the trench is supported by *bentonite slurry* during excavation, which is then replaced with a self-hardening cement–bentonite slurry to form the final *cutoff wall*.

Chemical grout Specialist *grouts* that are not based on cement suspensions and typically *gel* (solidify) on setting. These materials are most commonly silicate compounds or resins.

Coefficient of permeability See *Permeability*.

Cofferdam A temporary retaining wall structure used to exclude groundwater and surface water from an excavation.

Collector well A pumping technique used for water supply and *groundwater lowering* where a shaft is excavated below the groundwater levels. Lateral wells are drilled horizontally out from the shaft. Pumping water from the shaft draws water into the shaft from the lateral wells.

Conceptual model A theory-based description of groundwater and hydrogeological conditions that describes the relevant conditions that will affect the design and implementation of a *groundwater lowering* scheme. The elements of the conceptual model include geometry, boundary conditions, groundwater levels, and permeability values.

Cone of depression A depression in the *water table* or *piezometric level* that, in theoretically idealized conditions, has the shape of an inverted cone and develops radially around a well from which water is being abstracted. It defines the *radius of influence* of a well.

Confined aquifer An aquifer in which the groundwater is isolated from the atmosphere by an overlying (or confining) aquiclude and where the *piezometric level* is above the top of the aquifer. A confined aquifer is saturated throughout. Also known as an artesian or subartesian aquifer.

Consolidation settlements Ground settlements resulting from increases in vertical *effective stress* when groundwater levels are lowered or pore water pressures are reduced.

Constant head test A permeability test, typically carried out in a borehole or observation well, where water is added to or removed from the well at a constant flow rate. Observation of the change in water level in response to the flow rate can be used to estimate the *permeability* of the surrounding ground.

Construction dewatering See *Dewatering*.

Contiguous pile wall A type of *bored pile wall* used to form *cutoff walls* or retaining structures where successive concrete piles are bored in close proximity. In this application, the piles are not intended to intersect. Typically, a small gap remains between piles and may be sealed by grouting or other means.

Controlled waters A definition used in environmental legislation in England and Wales to describe all surface water, lakes, seas, and all groundwater.

Cutoff wall A very low permeability artificial barrier constructed to prevent groundwater from flowing into an excavation as part of a *groundwater exclusion* strategy.

Darcian velocity The mean flow velocity across a unit area as defined by *Darcy's law*. When divided by the *porosity*, the mean groundwater flow rate through the pores may be calculated.

Darcy's law The expression, developed by Henri Darcy in 1856, relating the flow rate through a porous medium to the *hydraulic gradient*, cross-sectional area of flow, and the *permeability* of the medium.

Deep well A groundwater abstraction well for the purpose of *groundwater lowering* (as opposed to a well for the purpose of water supply, which is termed a *borehole*).

Derogation The reduction in yield or other adverse effect on a water supply *borehole* or *spring* as a result of groundwater pumping by others.

Desk study The review of all available information (such as maps, aerial photographs, and historical records) as part of the *site investigation* process.

Development See *Well development*.

Dewatering A colloquialism for *groundwater lowering* that originated in America and is now widely used in Europe. Also known as construction dewatering.

Diaphragm wall A technique used to form *cutoff walls* or retaining structures where cast in situ concrete walls are formed within a trench supported by a slurry (most commonly but not always *bentonite slurry*) during excavation. Diaphragm walls are commonly known as D-walls.

Dipmeter A portable device for measuring the depth to water in a *well*, *borehole*, *standpipe*, or *piezometer*. Also known as a dipper or water level indicator.

Discharge The flow of water from a pump or a groundwater lowering system.

Drawdown The lowering of the *water table* (in unconfined aquifers) or the lowering of the *piezometric level* (in confined aquifers) as a result of the abstraction of groundwater. It is measured as the vertical distance between the original water table (or piezometric level) and the current level during pumping.

Drift Geologically recent superficial strata of sand, gravel silt, or clay (see also *Soil*). Excavations below the groundwater level in drift deposits may be unstable without adequate *groundwater control*.

Eductor See *Ejector*.

Effective stress The difference between the total stress (due to self-weight and any external loading) and the *pore water pressure* at a point in a soil mass. The effective stress is a vital controlling factor in the strength and compressibility behavior of soils.

Ejector A water jet pump that pumps water (and air) via a nozzle and venturi arrangement. When used in low-yielding wells, a vacuum can be developed if the top of the well casing is sealed.

Ejector well A small diameter groundwater lowering well pumped by an *ejector*.

Electro-osmosis A rarely used method of *pore water pressure control* applicable to very low permeability soils such as silts and clays. An electric potential is applied across the area of soil to induce groundwater flow from the anode to the cathode.

Equivalent well A conceptual large well, used in design to represent a groundwater lowering system that consists of a ring of many wells. By considering the real system as an equivalent well, simpler and more accessible methods of analysis can be used.

Falling head test A type of *variable head test*, typically carried out in a site investigation borehole or observation well and used to estimate *permeability*. At the start of the test, the water level in the well is raised rapidly and the rate of fall of the water level in the well is measured.

Filter pack Sand or gravel placed around a *well screen* to act as a filter. This allows water to freely enter the well while preventing movement of *fines* toward the well.

Fines The smaller particles in a soil stratum or analysis. Generally taken to refer to clay-, silt-, and fine sand-sized particles. See *Loss of fines*.

Fissures Natural cracks, fractures, joints, or discontinuities in *rock* that allow groundwater to pass. The extent and characteristics of fissures greatly influence the permeability of most rocks.

Flow, steady-state The flow regime when the magnitude and direction of the groundwater flow rate are constant.

Flow, transient-state The flow regime when the magnitude or direction of the groundwater flow rate is changing with time.

Flowing artesian conditions A special case of *confined aquifers*, where the piezometric level of the aquifer is above ground level. A *well* drilled through the confining bed into the aquifer will overflow at ground level without the need for pumping.

Fluidization due to upward seepage gradients A form of *base instability* that can occur when upward *hydraulic gradients* into an excavation cause a significant reduction in *effective stress* in the soil immediately beneath an excavation. If effective stress approaches zero, the soil can fluidize and lose all strength.

Formation level The final dig level of an excavation, after all digging and filling, but before any concreting or construction.

Free product Groundwater contamination present in an immiscible form (i.e., which does not mix with water).

Fully grouted piezometer A method sometimes used to install *rapid response piezometers* (such as *vibrating wire piezometers*), where the instrument or sensor is installed in the borehole surrounded by cement–bentonite grout, with no sand response zone.

Fully penetrating The case where a well penetrates the full thickness of an aquifer—the converse of *partially penetrating*.

Gap graded Granular materials, such as gravels, in which a specific range of particle sizes are entirely or substantially absent.

Garland drain A shallow ditch or trench for draining *surface water* or *groundwater* before it reaches an excavation.

Gel The process by which a *chemical grout* sets or hardens.

Geotechnical investigation *See Site investigation*.

Granular activated carbon A form of *groundwater treatment* where water is passed through tanks or beds of highly porous, chemically adsorbent, granular material (activated carbon). Activated carbon removes organic solutes by absorption onto the porous media. The activated carbon must be replaced periodically, as it may become saturated and unable to absorb.

Ground investigation The physical investigation of a site as part of the *site investigation* process. Ground investigation includes drilling and probing, in-site tests, and laboratory testing.

Groundwater Water contained in and flowing through the pores and fissures of soil and rock in the *saturated zone*. In this zone, the water body is essentially continuous, except for an occasional bubble of air. In legal documents, groundwater is sometimes described as water in underground strata.

Groundwater control Methods and techniques to control groundwater to allow stable excavations to be formed for construction purposes. The two main approaches are *groundwater exclusion* and *groundwater lowering*.

Groundwater exclusion The construction of artificial barriers to prevent groundwater from flowing into an excavation. The barriers can be formed from physical *cutoff walls*, or by reducing the permeability of the in situ soil and rock by *grouting* or other methods. See also *Cofferdam*.

Groundwater lowering The temporary reduction (or *drawdown*) of groundwater levels and pore water pressures around and below an excavation by some form of pumping, sufficiently to enable excavation and foundation construction to be carried out safely and expeditiously in water-bearing ground.

Groundwater treatment Processes applied to groundwater to remove contamination or otherwise improve water quality prior to disposal of the water.

Grout A pumpable material injected into the ground as part of the *grouting* process. Common types of grout are *cement-based grouts* or *chemical grouts*.

Grouting The process of controlled injection of material (termed *grout*), usually in a temporary fluid phase, into soil or rock, where it stiffens to improve the physical characteristics of the ground for geotechnical engineering reasons. The fluids injected are most commonly *cement-based grouts* or *chemical grouts*. Grouting is used to reduce the permeability of the ground to form a low-permeability barrier as part of *groundwater exclusion* methods.

Heterogeneous Nonuniform in structure or composition or composed of diverse elements—the converse of *homogeneous*.

Homogeneous Uniform in structure or composition—the converse of *heterogeneous*.

Horizontal directional drilling A form of directional boring, using steerable drilling techniques, to drill and install wells in a shallow arc to allow dewatering beneath inaccessible areas and without the need for vertical wells.

Hydraulic conductivity The hydrogeological term for *permeability*. In general, engineering literature uses permeability, whereas hydrogeological references use hydraulic conductivity.

Hydraulic failure *Base instability* caused by groundwater.

Hydraulic gradient The change in *total hydraulic head* between two points, divided by the length of flow path between the points.

Hydrogeology The study of groundwater or underground waters. Also known as groundwater hydrology.

Hydrological cycle The interlinked processes by which water is circulated from the oceans to the atmosphere, to the ground surface, and then returned to the oceans as surface water and groundwater flow.

Hydrology The study of water. In a civil engineering context, it is generally taken to be the study of surface waters.

Internal erosion Movement of soil particles in granular soils under the influence of flowing groundwater. Where this occurs in or around an excavation, it can lead to *base instability* or *hydraulic failure*.

Isotropic Having the same physical properties in all directions—the converse of *anisotropic*.

Jet grouting A form of *grouting* where the action of water/air/grout jets is used to disrupt existing soil structure and effect some in situ mixing, with and replacement by, injected *cement-based grout*. Jet grouting can be used to form low-permeability barriers as part of *groundwater exclusion* methods.

Jetting A method used to install wellpoints and, less commonly, deep wells or ejectors. A jet of high-pressure water is used to allow the penetration of a steel placing tube into the ground, within which the wellpoint or well is installed, following which the placing tube is removed. The jetting water returns to the surface as a *boil*, bringing with it the displaced soil.

Lamellae settlement tank A form of *groundwater treatment* used to remove suspended sand- and silt-sized particles from pumped water. The water is passed between closely spaced parallel inclined plates. This form of treatment is much more efficient at removing suspended solids than conventional *settlement tanks*.

Laminar flow A flow regime where flow is steady and continuous without any turbulence. It is characteristic of most groundwater flow through porous media and finely fissured rock formations—the converse of *turbulent flow*.

Leaky aquifer An aquifer confined by a low-permeability *aquitard*. When pore water pressures in the aquifer are lowered by pumping, water will flow (or leak) from the aquitard and recharge the aquifer. Also known as a semi confined aquifer.

Loss of fines The uncontrolled movement of *fines* carried by groundwater flow toward a well, sump, or soil face where filters are inadequate or absent. Also used to describe the washing of fines from samples recovered from water-filled boreholes during site investigation drilling.

Mix-in-place walls A technique used to form *cutoff walls* where low-permeability barriers are produced in situ by *soil mixing* where the mechanical action of an auger or cutter, in combination with *grout* injection into the mixing zone, creates a zone or block of treated material.

Numerical model A method of analyzing a groundwater *conceptual model* using computer software. The elements of the conceptual model are expressed as the geometry, boundary conditions, and numerical values of the problem. The resulting equations are solved numerically (often by iteration) using the software, generally run on a personal computer.

Observation well An instrument (a *standpipe*, *piezometer*, or unpumped *well*) installed into the ground in a selected location for the purpose of measuring the level of the *water table*, *piezometric level*, or *pore water pressure*.

Open area The proportion of the surface area of a *well screen* that is perforated or slotted and allows water to pass. Open area is normally expressed as a percentage.

Overbleed A commonly used colloquial term to describe residual groundwater seepages into an excavation, as might occur when a *perched water table* is penetrated. Overbleed normally describes low rates of seepage that are a nuisance, rather than causing significant flooding. Nevertheless, even at small rates of seepage, if *fines* are being continuously transported by the seepage, this should be counteracted immediately. The guiding principle is—do not try to stop the flow of seepage water, as this will cause a buildup of pore pressures but do prevent continuous *loss of fines*, perhaps by placing a suitable filter.

Partially penetrating The case where a well does not penetrate the full thickness of an aquifer—the converse of *fully penetrating*.

Particle size distribution The relative percentages by dry weight of the different particle sizes of a soil sample, determined in the laboratory by mechanical analysis (e.g., by sieving). Also known as PSD, soil grading, or sieve analysis.

Perched water table Water trapped (or "perched") in an isolated saturated zone above the water table. Perched water tables may exist naturally or may be caused when groundwater levels are lowered by dewatering; very low permeability silt or clay layers will tend to inhibit downward seepage of groundwater, trapping water above them. When an excavation is dug through a perched water table, water will enter the excavation as residual seepage or *overbleed*.

Permeability A measure of the ease with which water can flow through soil or rock. Also known as coefficient of permeability or *hydraulic conductivity*.

Permeable reactive barrier (PRB) A wall or trench that is filled with permeable materials designed to intercept a plume of groundwater contamination. The materials within the PRB are chemically reactive. As the contaminated water flows through the PRB, under the action of natural hydraulic gradients, contaminants will be removed to help reduce contaminant levels downgradient of the barrier.

Permeation grouting A form of *grouting* where *grout* is injected to fill or partially fill the permeable pores within a soil, without disturbing the structure of the soil. Permeation grouting can be used to form low-permeability barriers as part of *groundwater exclusion* methods.

Phreatic level The level at which pore water pressure is zero (i.e., equal to atmospheric pressure) and, thus, is at the base of the *capillary saturated zone*.

Piezometer An instrument, installed into the ground to act as an *observation well* to measure the pore water pressure or piezometric level in a specific stratum, layer, or elevation. This is achieved by having only the relevant section of the borehole exposed to the ground, the remainder of the borehole being sealed with grout or other very low permeability material. See also *Fully grouted piezometer*.

Piezometric level The level representing the *total hydraulic head* of groundwater in a *confined aquifer*. Also known as potentiometric surface.

Piping The washing out of fine particles from soils under the action of flowing groundwater to create preferential flow paths. This can lead to the creation of discrete open voids termed “pipes,” within which groundwater flow is concentrated. Where this occurs in or around an excavation, it can lead to *base instability* or *hydraulic failure*. See also *Internal erosion*.

Plane flow A two-dimensional flow regime in which flow occurs in a series of parallel planes. This type of flow occurs toward a pumped slot or long line of closely spaced wellpoints, when the direction of flow is perpendicular to the slot or line of wellpoints.

Plume A zone of contaminated groundwater that, under the action of *hydraulic gradients*, moves away from the source of the contamination.

Pore water pressure The pressure of groundwater in the pores of a soil, measured relative to atmospheric pressure. Positive and negative pore water pressures are greater and less than atmospheric, respectively.

Pore water pressure control The application of *groundwater lowering* to low-permeability soils. The fine-grained nature of such soils means that relatively little water drains from the pores and desaturation does not occur. Nevertheless, pumping from vacuum-assisted wells can reduce *pore water pressures*, thereby controlling effective stress levels and preventing instability.

Porosity The ratio of the volume of voids (or pore space) to the total volume of a soil sample or mass.

Predrainage The methods that lower groundwater levels in advance of excavation. Predrainage methods include *wellpoints*, *deep wells*, and *ejectors*. The converse of predrainage is *sump pumping*, where water is allowed to enter the excavation, from where it is removed.

Pump and treat A technique that can be used to remediate contaminated groundwater. In this approach, water is pumped from wells or sumps and is passed through a *groundwater treatment* system to remove contaminants prior to disposal.

Pumping test A method of in situ permeability testing involving controlled pumping from a well while the *discharge* flow rate and *drawdown* in the aquifer are recorded.

Radial flow A two-dimensional flow regime in which the flow occurs in planes, which converge on an axis of radial symmetry. This type of flow occurs toward an individual well or sump, when flow lines converge as they approach the well.

Radius of influence The radial distance outward from the center of the pumping well to the point where there is no lowering of the water table or reduction of the piezometric level—the edge of the *cone of depression*. Also known as the distance of influence.

Rapid response piezometer *Piezometers* that use specialist instruments (such as *vibrating wire piezometers*) where only a very small volume of water must flow into or out of the sensor in order to record a change in pressure. This type of piezometer is suitable to observe pore water pressure changes in silts, clays, and laminated soils. Rapid response piezometers may be installed as *fully grouted piezometers*.

Recharge Water that flows into an aquifer to increase or maintain the quantity of groundwater stored therein. Recharge may be from several sources: infiltration from surface waters; precipitation; seepage from other aquifers or aquitards; or *artificial recharge*.

Relief well A well installed within an excavation to act as a preferential pathway for flow to relieve any *pore water pressures* trapped beneath the excavation. As the excavation is deepened, the well will overflow, relieving pressures. Also known as a bleed well.

Rheology The study of the flow of materials, including fluids used in ground engineering, such as *grouts* and *bentonite slurry*.

Rising head test A type of *variable head test*, typically carried out in a borehole or observation well, used to estimate *permeability*. At the start of the test, the water level in the well is lowered rapidly and the rate of rise of the water level in the well is measured.

Rock A geological deposit formed from mineral grains or crystals cemented together—this is distinct from uncemented *soil* or *drift*.

Rock grouting A form of *grouting* where *grout* is injected to fill or partially fill *fissures*, fractures, or joints in a rock mass, without creating new fractures or opening existing fractures. Rock grouting can be used to form low-permeability barriers as part of *groundwater exclusion* methods.

Running sand A colloquial term for the conditions when saturated granular soils are so unstable that they are unable to support a cut face or slope and become an almost liquid slurry. Running sand is not a type or property of the material; it is a condition in which a granular soil can exist under unfavorable seepage conditions. Effective *groundwater lowering* can change running sand into a stable and workable material.

Saline intrusion The movement of saline water into a freshwater aquifer as a result of *hydraulic gradients*. Problems of saline intrusion can be created or exacerbated as a result of pumping from wells.

Saturated The condition when all the pores and fissures of a soil or rock are completely filled with water—the converse of *unsaturated*.

Saturated zone The zone of *saturated* soil or rock, all parts of which are in hydraulic connection with each other. In a confined aquifer, the saturated zone comprises the entire aquifer. In an unconfined aquifer, the saturated zone is the area below the water table, plus the *capillary saturated zone* above the water table.

Secant pile wall A type of *bored pile wall* used to form *cutoff walls* or retaining structures where successive concrete piles are bored in close proximity. In this application, alternate piles are drilled at spacings of less than one pile diameter so that they intersect and form a continuous wall.

Settlement lagoon A large lagoon or pit through which the pumped discharge is passed to settle out sand- and silt-sized particles prior to disposal.

Settlement tank A small self-contained tank (often containing baffles) through which the pumped discharge is passed to settle out sand-sized particles prior to disposal. Such relatively small tanks are ineffective at settling out silt-sized particles due to the short retention time. See also *Lamellae settlement tank*.

Sheet piling A technique used to form *cutoff walls* or retaining structures where a series of interlocked steel sections (typically of a "Z" or "U" profile) are driven or pushed into the ground to form a continuous barrier.

Shoestring A colloquial term used to describe a narrow but perhaps extensive lens of soil markedly different from the stratum of soil within which it is encompassed. The term is usually applied to a lens of permeability significantly greater than the surrounding soil.

Siphon drain A specialist pumping technique occasionally used for *groundwater lowering* that allows water to be pumped by gravity (without the need for external energy input) via permanently primed siphon pipes.

Site investigation The overall process of obtaining all relevant information about a site to allow the design and construction of a civil engineering or building project. Elements of site investigation include the *desk study* and the *ground investigation*.

Slimes A mining term used to describe the residual waste products remaining after washing mined mineral extracts. Sometimes referred to as washings or tailings.

Slug test A *variable head test*, typically carried out in a borehole or observation well, used to estimate permeability. At the start of a slug test, the water level in the well is subjected to a rapid change in water level. This is achieved by inserting or removing a heavy rod into the well. When the rod passes the water level, it will displace water and hence rapidly change the water level. Observation of the subsequent rate of change of water level in the borehole as the head change dissipates can be used to estimate the *permeability* of the surrounding ground. See also *Falling head test*; *Rising head test*.

Slurry trench wall A type of *cutoff wall* formed by the excavation of a trench supported by a slurry (most commonly but not always *bentonite slurry*) during excavation.

Soil The term used in civil engineering to describe uncemented deposits of mineral (and occasionally organic) particles such as gravel, sand, silt, and clay—this is distinct from cemented *rock*. Excavations below the groundwater level in soils may be unstable with adequate *groundwater control*. See also *Drift*.

Soil–bentonite wall A type of *slurry trench wall* where the trench is supported by a *bentonite slurry* during excavation, which is then replaced with a mixture of soil and bentonite to form the final *cutoff wall*.

Soil mixing A technique used to form *cutoff walls* by the *mix-in-place* method where the existing soil structure is disrupted by mechanical tools (e.g., augers or cutters) and then the soil is mixed in situ with injected grout.

Solution grout *Grouts* that do not contain solid particles and generally *gel* (solidify) on setting. The most common form of solution grouts are *chemical grouts*.

Specific yield The storage coefficient in an unconfined aquifer.

Spring A natural outflow of groundwater at the ground surface. In rural areas, springs may be used as water supplies.

Standpipe An instrument, typically consisting of a perforated casing or pipe, installed into the ground to act as an *observation well*. A standpipe is open to water inflows from all strata that it penetrates and may give a water level reading that is a hybrid of levels in different aquifers. A *piezometer* should be used to determine water levels or pore water pressures in a specific stratum.

Storage coefficient The volume of water released by gravity drainage from a volume of aquifer. Normally expressed as a dimensionless ratio or percentage. Also known as storativity.

Storage release The release of water from storage in an aquifer. During the early stages of pumping (the first few hours or days), storage release can significantly increase the discharge flow rate above the steady-state value. Storage release is most significant in unconfined aquifers of high permeability.

Stratum A geological term for a layer of any deposited *soil* or *rock*. The plural of stratum is strata.

Subdrain A perforated or open-jointed pipe sometimes found immediately beneath sewers or pipelines that were laid in trenches. During construction, the subdrain is laid in the trench just ahead of the main pipe, to carry water away from the working area.

Submersible pump A pump designed to operate wholly or partly submerged in water. The most common forms of submersible pumps are electrically powered. Special slimline submersible pumps are suitable for use in *wells* or *boreholes*.

Sump A pit, usually located within an excavation, in which water collects prior to being pumped away.

Sump pump A robust pump, capable of handling solids-laden water, used to pump groundwater and surface water from sumps.

Sump pumping A method of *groundwater lowering* that allows water to enter the excavation, from where it is pumped away. In some circumstances, the flow of water into the excavation may have a destabilizing effect, and the use of *predrainage* methods (in preference to sump pumping) may reduce the risk of instability.

Surface water Water contained in rivers, streams, lakes, and ponds.

Suspension grout *Grouts* that comprise solids suspended in water without being dissolved. The most common form of suspension grouts are *cement-based grouts*.

Total hydraulic head The potential energy of water due to its height above a given level. The total head controls the height to which water will rise in a piezometer. The total head at a given point is the sum of the elevation head (i.e., the height of the point above an arbitrary datum) and the pressure head (i.e., the height of water above the point recorded in a standpipe piezometer). Also known as total head or total hydraulic potential.

Transmissivity A term used in *hydrogeology* to describe the ease with which water can flow through the saturated thickness of the aquifer. Transmissivity is equal to the product of *permeability* and saturated aquifer thickness.

Tube à manchette (TAM) A technique used as part of *permeation grouting* where special grouting tubes are installed into boreholes. The TAM pipes comprise polyvinyl chloride or metal pipes with grout ports drilled in them at specific intervals. Rubber sleeves cover the grout ports, and the TAM pipes are sealed into the boreholes with weak sleeve grout. *Grout* can be injected into the TAM pipes via special packers, which allow grout to be injected into the ground from specific levels in the boreholes.

Turbulent flow A flow regime where the flow lines are confused and heterogeneously mixed. It is typical of flow in some surface water bodies—the converse of *laminar flow*.

Unconfined aquifer An aquifer whose upper surface is not confined by an overlying impermeable stratum and whose upper surface is directly exposed to atmospheric pressure. Also known as a water table aquifer.

Underdrainage A method of *groundwater lowering* that uses soil layering to advantage by pumping directly from a deeper, more permeable stratum, in preference to the overlying lower-permeability layers. Pumping from the deeper layer promotes vertical downward drainage from the lower-permeability layer into the stratum being pumped.

Uniformly graded Containing a narrow range of particle sizes so that most of the particles are of similar size—the converse of *well graded*. Also known as poorly graded.

Unsaturated The condition when the pores and fissures of a soil or rock are not completely filled with water and may contain some air or other gases—the converse of *saturated*.

Unsaturated zone The zone of *unsaturated* soil or rock. In an unconfined aquifer, the saturated zone is the area above the *capillary saturated zone*. Within the unsaturated zone, the forces of surface tension render the water pressure negative (i.e., lower than atmospheric).

Vadose zone See *Unsaturated zone*.

Variable head test A permeability test, typically carried out in a borehole or observation well, where at the start of the test, the water level in the well is subjected to a rapid (ideally instantaneous) change in water level. Observation of the subsequent rate of change of water level in the borehole as the head change dissipates can be used to estimate the *permeability* of the surrounding ground. See also *Falling head test*; *Rising head test*; *Slug test*.

Vibrated beam walls A technique used to form *cutoff walls* that involves successive overlapping insertions of a steel probe or mandrel into the ground, followed by the injection of *grout* (typically a cement–bentonite slurry) into the slot left behind by the mandrel as it is withdrawn.

Vibrating wire piezometers *Piezometers* where vibrating wire pressure transducers are used to measure pore water pressures. Vibrating wire instruments contain a metal diaphragm in hydraulic connection with the groundwater. A taut wire is stretched between the diaphragm and a stable datum. When the instrument is read, it is "plucked" by passing a controlled frequency electrical pulse along it. The taut wire resonates at a frequency related to its tension, which can be related to the defection of the diaphragm and hence the water pressure on the diaphragm. See also *Rapid response piezometer.*

Void ratio The ratio of the volume of voids to the volume of solids in a soil mass.

V-notch weir A thin plate *weir* where a V-shaped notch of a specified angle is cut. Calibration charts allow the flow rate to be estimated from measurements of the height of water flowing over the weir.

Water table The level in an unconfined aquifer at which the pore water pressure is zero (i.e., atmospheric). See also *Phreatic level.*

Weir A structure used to control the flow of water, often so it can be measured. See also *V-notch weir.*

Well A hole sunk into the ground for the purpose of abstracting water. Wells for groundwater lowering purposes are generally categorized by their method of pumping as *deep wells*, *ejector wells*, or *wellpoints*. In water supply terminology, a well is often taken to mean a large diameter shaft, as may be dug by hand in developing countries. A smaller diameter well, constructed by a drilling rig, is termed a *borehole*.

Well casing The unperforated section of the *well liner*, installed at depths where any groundwater present is to be excluded from the well (e.g., if several aquifers exist but drawdown is not required in all of them). Also known as plain casing or solid casing.

Well development Increasing or maximizing the yield of a well by removing drilling residue and fine particles from the well and the aquifer immediately outside the well. Normally carried out following well construction but prior to the installation of pumping equipment.

Well liner A generic term for *well casing* and *well screen*.

Well loss The head loss (or additional drawdown inside the well) that occurs when water flows from the aquifer, through the *well screen* and *filter pack*, and into the well itself.

Well screen The perforated or slotted section of the well liner installed to allow water to enter a well where it penetrates an *aquifer.* Also known as perforated casing or slotted casing.

Well graded Containing a wide range of particle sizes—the converse of *uniformly graded*. A soil formed from a mixture of various proportions of gravel-, sand-, silt-, and clay-sized particles would be described as well graded.

Wellpoint A small diameter shallow well for groundwater lowering purposes. Normally installed by jetting and pumped by a suction pump of some kind.

Yield The discharge flow rate of a well. The yield is generally taken to be controlled by the well, rather than the pump—a high-yielding well would produce a low flow rate if an undersized pump were installed. Also known as well yield.

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