THE APPLICATION OF ENGINEERING GEOLOGY TO DAM CONSTRUCTION

or what experience has taught us

Otto Horský Pavel Bláha

“A WELL-BEHAVED DAM AND RESERVOIR”
Water for 15 million people
Angat, Philippines

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or “What Experience Has Thaught Us”

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The English book edition in hardback was published in 2011 in a print run of 1,000 copies and is already 70 % sold out. This book edition on a CD is supplemented by certain other subchapters and knowledge. The book was also published in hardback in the Czech language (in 2009) and is practically all sold out. A supplemented book edition in the Czech language on a CD will also be released in 2014.

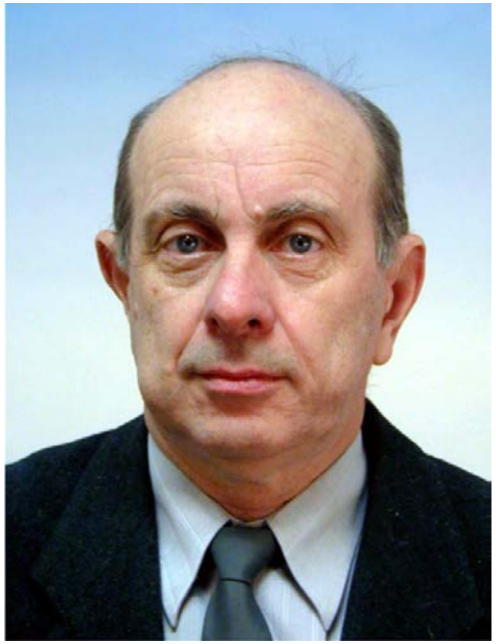
Last, but not least, we offer our warmest thanks to our wives, Marie and Oldřiška, for their understanding and for creating the environment that enabled us to work on this book and for their help in eliminating formal mistakes from the text.

The authors would be grateful to readers for any comments on the content of this book. These can be sent to us at our electronic addresses: horsky@horsky.org or blaha@geotest.cz.

Ing. Otto Horský, CSc. (1938, Prostějov)

Otto Horský graduated in geology at the School of Mines (VŠB) in Ostrava in 1961. In 1978 he defended his CSc. (PhD) thesis in engineering geology at the same school. He began his work with the company Geologický průzkum Brno (renamed in 1968 to Geotest Brno) in 1960. His major professional tasks as an engineering geologist include the survey for the dam and pumped storage hydroelectric plant at Dalešice and later surveys for dams in Cuba and over twenty other dams in the Czech Republic and elsewhere. Special works include remediation of emergency situations of dams, the most significant of which is remediation of the Orava reservoir banks in Slovakia. Between 1974 and 1976 he worked as chief of a technical office in Peru and carried out engineering-geological and geotechnical advisory activity for dams and hydroelectric power plants. He prepared and signed for the former Czechoslovakia a contract for a geotechnical survey for the underground powerhouse Mantaro – Restitución and called attention to a rock slide at the site of the already completed Tablachaca Dam on the River Mantaro. On this basis, Czechoslovak experts prepared a project of remediation of the slide, which was subsequently implemented. As an expert, he participated in the geotechnical survey work for the extension of the hydropower potential of the Machu Picchu hydroelectric power plant; he visited a number of other dams in Peru and proposed technical designs. From 1978 to 1982 he served in Cuba as chief adviser for engineering geology to the Ministry of Construction. Then, from 1984 to 1988 he was in charge of an engineering-geological survey for a PSHEP in the Escambray Mountains in Cuba and, at the same time, coordinated the work of another Czechoslovak team in Oriente Province. From 1991 to 1994 he worked in Spain as General Director of a Czech-Spanish company specializing in geology and ecology. Since his return to the Czech Republic, he has continued with the design and evaluation of geological projects. In a professional capacity, he visited Mexico, Chile and Cuba, and carried out other work in Spain and in the Canary Islands with Geotest. From 1971 to 1991 he was seconded to the Technical University in Brno. In 1985 he was visiting lecturer in engineering geology at the School of Mines in Ostrava and while he was adviser to the Ministry of Construction in Cuba he also gave lectures and training to Cuban specialists in engineering geology. He has also been invited to give lectures at universities in Spain, Mexico and Peru. He has more than 150 professional publications, including several books, more than a hundred of which have been published in the former Czechoslovakia or in the Czech Republic and the others abroad in Spain, Brazil, Peru, Russia, the Netherlands, Australia, Portugal, Greece, Thailand, Ethiopia and Cuba. In addition, he has compiled more than 200 final reports and appraisals, largely for hydraulic structures not only in the Czech Republic and Slovakia, but also in Spain, Cuba and Peru. He works as a reviewer for certain professional journals, for example, for EGRSE or for Environmental Geology. Since 2010 he has been collaborating as an expert with CREA and Geotest for the project of the Bawanur dam in Iraq – Kurdistan Region.

Assoc. Prof. RNDr. Pavel Bláha, D.Sc. (1944, Protivín)



Pavel Bláha completed his studies in applied geophysics in the Faculty of Natural Sciences at Charles University in Prague in 1966. In 1990 he defended his CSc. (PhD) thesis on “Geoacoustic Methods for the Study of Slope Failures”. In 1992 he habilitated (Associate Professor) with the thesis on the application of geophysical methods to the survey of slope Failures and a lecture on seismic tomography in the near-surface geology in the Faculty of Mining and Geology (FMG) of the VŠB-Technical University of Ostrava (VŠB – TUO) and in 1998 he defended his doctorate on “Engineering Geophysics of Slope Failures in Relation to Mining and Building Geotechnics”. After serving for a short period from 1966 to 1971 as a lecturer at the FMG, he joined Geotest in Brno as a geophysicist where he still works. His first post was as chief of a geophysical team, then he was appointed chief of the Ostrava Branch of Geotest (to 2006) and, ultimately, adviser – consultant. The focus of his professional career has been the application of geophysical methods to the surveying of dam sites and pumped storage hydroelectric plants and, specifically, the application of geophysics to the monitoring and measurement of slope failures. In the Czech Republic, such constructions were the Dalešice and Josefův Důl dams, the Dlouhé Stráně PSHEP, the Dalešice nuclear power plant and the Brno-Ostrava motorway. The slope failures included particularly the landslides in the Carpathian System and the slope failures relating to mining activity. He is the author of over 455 certified survey reports and over 80 research reports that passed through external examination. During his professional career, he has given lectures on selected chapters about the application of geophysical methods in near-surface geology at universities in Ostrava, Brno, Prague, Bratislava and Tashkent. During the 48 years of his professional career he has worked in 20 countries worldwide, mainly in Central Asia, Spain, Albania, Cuba, Mongolia, Iraq and the Philippines. He is the author of over 200 articles in professional journals and in the proceedings of specialist conferences of which more than 70 foreign countries (Albania, Bosnia and Herzegovina, Bulgaria, Ethiopia, France, Italy, Kyrgyzstan, Malaysia, Germany, the Netherlands, Norway, Poland, Romania, Russia, Greece, Scotland, Slovakia, Spain, Tajikistan, Thailand and Uzbekistan). Our works have been referenced in more than 250 professional publications. He published seven monographs as an author or co-author. He works as a reviewer for certain professional journals in the Czech Republic, Slovakia, Poland and Uzbekistan. He is chairman of the editorial board of the journal “EGRSE”, the journal of the Czech Association of Geophysicists, and a member of the Council of the Association of Czech Geophysicists, a member of the editorial board of the journals “Krystalinikum” and “Geotechnika”. He is an academic supervisor of PhD research students (geophysics and GIS) at the FMG and the Faculty of Civil Engineering of the VŠB – TUO. He is also a member of the Departmental Council for Geology at the FMG of the VŠB – TUO and serves as a member of the Commission responsible for granting certificates of “Professional Competence” at the Czech Ministry of the Environment. He has been working as a member of steering committees of conferences and workshops including HYDRO 2011.

About the Book

The Application of Engineering Geologyto Dam Construction, or "What Experience Has Taught Us"

The present publication is intended for those interested in the procedures used and the problems encountered in carrying out engineering-geological surveys for the design and construction of dams. The intention is to provide a basis for technical dialogue between engineering geologists, geophysicists, hydrogeologists, geotechnicians and other specialists on one hand, and investors, planners and designers on the other. The book should also be of use to those involved in the operation and management of dams and reservoirs. The book has been designed so that it can be used as a text in colleges and universities. It summarizes the results of work carried out by the authors, one an engineering geologist and the other a geophysicist, at different dam sites in the Czech Republic and abroad. Both authors have had a long career serving as experts or consultants in engineering geology, and have visited many dam projects in other countries. This book is the outcome of their combined theoretical and practical experience.

The book consists of an introduction, a conclusion, and eight technical chapters. In all of them, the authors have placed special emphasis on practical examples, since the scope of the tasks involved in carrying out and interpreting the results of engineering-geological surveys can best be understood by using case studies to illustrate the techniques used and the problems encountered. An effort has been made not to become too deeply involved in the theoretical background to the subject. Rather than dictating rigorous rules and standards, an attempt has been made to illustrate scenarios that might be encountered within the planning of a project and the operation of a dam. The authors have especially drawn attention to circumstances, the neglect of which can lead to significant increases in the costs of dam construction and that, in some cases, may make the operation of a dam impossible.

In the introductory chapters the basic criteria for the choice of dam design are discussed in relationship to the characteristics of the selected site. The roles of geology, geomorphology, climate, ecology, and other features that affect the selection of the site for a dam are discussed. The basic principles and tasks of engineering-geological surveys for dam sites are described. Importance is placed on the division of the survey into stages corresponding to the design stages, as well as on the requirement that the survey must take account of all factors that affect safety during dam construction. Great attention is paid to the strategy of the survey, including a detailed list of the separate tasks involved. An important task of the book has been to draw systematic attention to the technical specifications and the planning of survey work, in which the investor, the designer, or simply the client who contracted the survey formulates the basic requirements that the survey should meet. The procedures used for engineering-geological mapping of the area of a hydro-engineering structure are described in detail paying special attention to the dam site itself. The method used to compile special-purpose engineering-geological maps is also explained. The scope of both preliminary and detailed hydrogeological surveys is described together with the methods used to check the condition and correct function of hydrogeological boreholes using geophysical techniques, TV cameras, etc.

In the second part of the book, the applications of geophysical measurements at various stages of a survey are explained. The use of particular geophysical methods to solve specific problems is illustrated by case studies. Emphasis is placed on the importance of collaboration between the engineering geologist and the geophysicist in defining the problems to be solved, and planning and carrying out the tasks required by the survey. A description of the scope of direct survey work follows, paying special attention to the study of the site directly affected by the construction of the dam itself and to the delineation of the area to be covered by the preliminary and detailed surveys. The methodology used for comprehensive documentation of exploratory workings is assessed in great detail. The authors also pay particular attention to the general principles governing a geotechnical survey, and the methods used to carry it out. The main types of field and laboratory geotechnical tests are described. An important sub-chapter deals with the systematic correlations and the empirical relationships between the mechanical and physical properties of rocks.

In the last section of the book, the procedures used for engineering-geological surveys of the backwater areas of dams are described. Special attention is given to the modification of the banks of water reservoirs by processes of abrasion, suffosion and landslides. The methods used to map and monitor these effects are described and examples of remedial procedures used to counteract these geodynamic processes are given.

In this book the authors have done their best to describe the procedures used for the engineering-geological and geotechnical surveys of dam sites and backwater areas, drawing attention to the problems that can be encountered in the design and construction of dams. The recommended working procedures have been described systematically. All the problems analysed in this book and the methods used to solve them are illustrated by practical examples. The authors trust that their experience of engineering geology and its applications gained over many years and described in this book will be of interest not only to experts, but also to students and the wider public interested in the global management of water resources.

the Authors

A propos du livre

Prospection d'ingénierie géologique pour les barrages,

ou 'Ce qui nous a aussi instruit'

Le présent ouvrage a été élaboré avec l'intention d'informer les personnes intéressées par la problématique de la prospection lors de la conception et de la construction des barrages. Il peut utilement servir à la compréhension mutuelle entre les investisseurs et les concepteurs d'une part et les ingénieurs géologues, géophysiciens, hydrogéologues, géotechniciens et autres spécialistes du sujet, d'autre part. Un autre groupe auquel nous nous adressons également sont les employés des organismes qui gèrent l'exploitation des barrages. L'ouvrage peut aussi être utilisé dans l'enseignement universitaire. Ce livre est le résultat de l'expérience et de travaux accomplis d'un ingénieur géologue et d'un géophysicien sur différents barrages en Republique Tchèque et sur de nombreux barrages à l'étranger. Les deux auteurs y ont travaillé en tant qu'experts, consultants ou y étaient présents comme visiteurs et observateurs.

L'ouvrage comporte une introduction, une conclusion ainsi que huit chapitres techniques. Dans chacun d'eux, nous avons mis l'accent particulier sur des exemples pratiques et sur des solutions de cas concrets, considérant que c'est la meilleure façon de rendre cette large problématique la plus compréhensible. Pour la même raison, nous nous sommes efforcés d'éviter les analyses théoriques, les thèses et les normes. En revanche, nous mettons en avant des situations concrètes, rencontrées lors de la préparation des projets et de l'exploitation des barrages. Notre objectif était également de mettre en évidence les cas spécifiques dont la négligence pourrait mener à une augmentation du prix de la construction du barrage ou à l'impossibilité de son exploitation dans le sens initialement désiré.

Les premiers chapitres résument les critères de base concernant la conception des barrages en relation avec les facteurs qui déterminent le choix de l'emplacement et du type de barrage. Nous évaluons les caractéristiques géologique, morphologique, climatique, écologique ainsi que d'autres facteurs qui peuvent influencer le choix de l'emplacement. Nous définissons ensuite les tâches spécifiques à traiter ainsi que les principes de la prospection des barrages en ingénierie géologique. On traite ici également la répartition de la prospection en étapes qui devraient coïncider avec les autres étapes du projet, tout en maintenant l'efficacité de la prospection et les impératifs de sécurité de la construction. Une attention particulière est apportée à la stratégie de la réalisation de la prospection ainsi qu'à l'élaboration d'une liste détaillée des tâches individuelles. Les caractéristiques des spécifications techniques et du projet des travaux de prospection sont des points importants où l'investisseur, le concepteur ou le client formulent les exigences de base qui devraient être traitées par la prospection. On définit les tâches de l'ingénierie géologique concernant la cartographie de la région et tout particulièrement de l'endroit où sera placé le barrage. On décrit ensuite la méthode pour élaborer une carte spécifique d'ingénierie géologique. On montre également l'utilité de la prospection hydrogéologique (préliminaire et détaillée) ainsi que les possibilités des contrôles de justesse et de fonctionnalité des forages hydrogéologiques par des méthodes géophysiques, des caméras TV etc.

La seconde partie du livre présente l'utilisation des méthodes géophysiques dans les différentes étapes de la prospection. Nous y évaluons les possibilités d'application des méthodes individuelles de géophysique face aux problèmes spécifiques. Nous attirons l'attention sur la nécessité d'une collaboration entre l'ingénieur géologue et le géophysicien pour définir les tâches à résoudre lors de la prospection et de l'interprétation. On aborde ensuite l'évaluation de l'étendue des travaux de prospection en tenant compte de l'étude de l'emplacement concerné par la construction du barrage, de la délimitation du domaine et de l'étendue de la prospection préliminaire et détaillée.

La méthodologie d'élaboration d'une documentation complexe des travaux de prospection est traitée dans le moindre détail. Une très grande attention est apportée à l'évaluation des critères généraux de la prospection géotechnique ainsi qu'à ses types et ses méthodes de base. Nous avons également introduit les différents types de tests géotechniques, aussi bien de terrain que de laboratoire. Un important sous-chapitre est consacré aux corrélations et aux rapports entre les propriétés mécaniques et physique des roches.

Dans la dernière partie nous décrivons la prospection par ingénierie géologique des zones inondables d'un barrage. Une attention particulière est apportée aux modifications des berges par abrasion et affaissement. Sur base de nos connaissances, nous proposons des solutions méthodologiques et le monitoring du développement des processus géodynamiques en jeu.

Dans cet ouvrage nous avons tenté d'aborder certains problèmes associés à la prospection lors de la conception et de la construction des barrages et de proposer des solutions possibles et des procédés de travail adaptés. Chacun des problèmes décrits est documenté et illustré par des cas réels, rencontrés en pratique. Nous espérons que ce recueil basé sur notre longue expérience sera une contribution utile non seulement pour les spécialistes, mais également d'une lecture agréable pour un plus large public qui s'intéresse à la problématique traitée ici.

Les auteurs

Prólogo

La aplicación de la ingeniería geológica a la construcción de presas

o "Lo qué hemos aprendido también"

El libro es fruto de los trabajos realizados por un ingeniero-geólogo y un geofísico en varias presas en la antigua Checoslovaquia y muchas más en otros países, en las que actuaron como responsables de las investigaciones o como asesores.

El libro está destinado a los interesados en la problemática de las investigaciones a efectuar durante el proyecto, construcción y explotación de presas y embalses. Puede ser útil para la comprensión y entendimiento entre las entidades que los promueven y los proyectistas, y entre los ingenieros, geólogos, geofísicos, hidrogeólogos, geotécnicos y otros especialistas. Será igualmente útil a cuantos técnicos intervienen en el mantenimiento de presas y en la explotación de embalses. También puede servir como texto en centros de enseñanza superior.

El libro consta de la introducción, conclusiones y ocho capítulos. En todos los capítulos se exponen casos reales, considerando que la mejor forma para comprender la amplia problemática de las investigaciones es la explicación de casos concretos. Se ha pretendido reducir la parte teórica y presentar casos concretos, que hemos encontrado en las fases de proyecto, construcción y explotación, antes que exponer las normativas y las teorías. Objetivo principal es incidir en casos, en los que errores en las investigaciones originaron sobrecostes en la construcción de la presas, problemas durante la explotación, e incluso siniestros y daños catastróficos.

En los primeros capítulos se exponen los criterios básicos para el diseño de la presa, los factores que determinan la selección de su ubicación y su tipología. Se analizan los aspectos geológicos y tectónicos, morfológicos, climatológicos, ecológicos y cuantos tienen importancia para la selección de la cerrada. Continúan con la metodología para las investigaciones de Ingeniería Geológica. Se considera muy importante que las fases de las exploraciones estén de acuerdo con las fases de del proyecto, para mejorar la efectividad de los trabajos y cumpliendo con todos los aspectos relacionados con la seguridad de la obra. Se presta especial atención a la técnica de las exploraciones de Ingeniería Geológica, definiendo con detalle todas las tareas fundamentales a realizar. Es de gran importancia definir las características técnicas del proyecto, de acuerdo la entidad promotora de la presa y embalse, el proyectista y el geólogo ingeniero, que en definitiva establecen los objetivos de las investigaciones y las posibles soluciones. Se continúa con un capítulo en el que se describe con detalle el levantamiento a efectuar sobre el área de interés y en especial sobre la cerrada, ubicación de la presa. Se describe minuciosamente el proceso de cartografía y metodología para presentar todos los datos geológicos, hidrogeológicos y geotécnicos. Se describe el modo y número de investigaciones hidrogeológicas necesarias, incluido el control de sondeos y pozos hidrogeológicos por métodos geofísicos, cámaras de TV, etc.

En la segunda parte del libro, se exponen los estudios geofísicos básicos en las diferentes fases de las investigaciones. Se definen las posibilidades de los diferentes métodos de prospección geofísica para resolver problemas concretos. Se considera muy importante la estrecha colaboración entre el ingeniero geólogo y el geofísico durante todas las fases de la investigación: definición de los trabajos, realización y presentación de los resultados. Se continúa con el análisis del número e intensidad de exploraciones directas a efectuar. Especial atención se dedica al estudio del emplazamiento de la presa y del área afectada por las obras. Se define la extensión del área a estudiar en cada etapa de las investigaciones. En un capítulo se exponen los métodos de presentación de la documentación de todos los trabajos de exploración de Ingeniería Geológica y Geotecnia. Se presta mucha atención a la descripción de los criterios y procedimientos básicos en las investigaciones geotécnicas. Se reseñan los ensayos de campo y de laboratorio necesarios para definir los parámetros geotécnicos que se precisan para el proyecto de la presa. Se califican con detalle las correlaciones y enlaces entre los parámetros mecánicos y físicos de las rocas.

En la última parte del libro se describen las investigaciones de Ingeniería Geológica a realizar en el vaso del embalse para prever los procesos morfológicos de erosiones e inestabilidades de ladera. Se proponen modos de solucionar los posibles problemas y monitorizar su desarrollo.

En nuestro libro se intenta hacer un esbozo de los problemas relacionados con las investigaciones de ingeniería geológica durante el proyecto y construcción de presas y proponer soluciones y procedimientos de los trabajos. Todos los capítulos se documentan con casos concretos. Consideramos que el resumen de nuestra experiencia práctica de muchos años puede ser una ayuda e información no sólo para los profesionales, sino también para el público interesado por estas estructuras y su problemática.

Los autores

Über das Buch

Ingenieurgeologische Erkundung für Staudämme

oder „Was uns auch belehrte“.

Vorliegende Publikation ist allen bestimmt, die sich für die Erkundungproblematik bei Entwurf und Bau von Staudämmen interessieren. Sie kann dem gegenseitigen Verständnis von Investoren und Projektleitung dienen und dieses auch zwischen Ingenieurgeologen, Geophysikern, Hydrogeologen, Geotechnikern und weiteren Spezialisten fördern. Eine weitere Gruppe, die wir ansprechen wollen, sind Mitarbeiter der staudammbetreibenden Organisationen. Dieses Buch kann gleichwohl als Lehrtext beim Hochschulstudium dienen. Es ist das Resultat der Arbeit und der Erfahrungen eines Ingenieurgeologen und eines Geophysikers beim Bau verschiedener Staudämme im Inland sowie zahlreicher Staudämme im Ausland, wo beide Autoren entweder als Problemlöser oder als Berater wirkten oder diese Bauten besuchten.

Diese Abhandlung besteht aus einer Einführung, acht Fachkapiteln und einer Zusammenfassung. Wir legen besonderen Wert auf Praxisbeispiele, da man an konkret gelösten Aufgaben am besten die ganze Bandbreite der Problematik aufzeigen kann. Mehr noch als auf theoretische Analysen, Normen und Richtlinien verweisen wir auf die Umstände, denen man während der Durchführung eines Projektes respektive während der Bauphase begegnen kann. Unser Ziel ist besonders der Verweis auf solche Gegebenheiten, deren Nichtbeachtung entweder zu höheren Baukosten führen oder die ursprünglich beabsichtigte Art der Nutzung unmöglich macht.

Die ersten Kapitel behandeln die Grundkriterien der Projektarbeit in Zusammenhang mit bestimmenden Faktoren für die Wahl des Ortes und des Typs der Staumauer. Bewertet sind geologische, morphologische, klimatische und ökologische Aspekte, sowie weitere – den Standort beeinflussenden – Faktoren. Weiterhin werden Grundaufgaben und Prinzipien der ingenieurgeologischen Erkundung für den Bau von Staudämmen definiert. Als wichtig betrachten wir das Aufteilen dieser Erkundung in Etappen, die auch den Etappen der Projektarbeit entsprechen. Weiterhin legen wir Wert auf die Effektivität der Erkundung hinsichtlich der Einhaltung von Sicherheitskriterien; große Aufmerksamkeit widmen wir der Erkundungstrategie mit detaillierter Auflistung einzelner Aufgaben. Ein wichtiger Bestandteil ist die Beschaffenheit der technischen Erörterung und der Erkundungsarbeiten, bei der der Investor, die Projektleiter bezeihungsweise Auftraggeber ihre Anforderungen hinsichtlich der Problemlösungen definieren. Es folgt eine definierte Aufgabe der ingenieurgeologischen Kartierung des Interessengebietes und eine detaillierte Beschreibung des Staudammstandortes. Des Weiteren wird die Zusammenstellung zweckgebundener ingenieurgeologischer Karten beschrieben sowie der erforderliche Umfang der hydrogeologischen Erkundung (vorläufiger und detaillierter). Weiterhin werden Möglichkeiten aufgezeigt, hydrogeologische Bohrungen hinsichtlich Richtigkeit und Funktionsfähigkeit mittels geophysikalischer Methoden, Fernsehkamera oder ähnlichem zu kontrollieren.

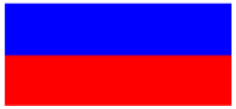
Im zweiten Teil des Buches werden die Aufgaben geophysikalischer Messung in verschiedenen Etappen der Erkundung aufgeführt. Dabei werden Möglichkeiten einzelner geophysikalischer Methoden zur Lösung konkreter Probleme analysiert, wobei Wert auf die Zusammenarbeit des Ingenieurgeologen und des Geophysikers bei der Aufgabendefinition, die zur Problemlösung führt und der anschließenden Projektarbeit gelegt wird. Weiter wird der Umfang der direkten Erkundungsarbeiten analysiert; an dieser Stelle beschäftigen wir uns insbesondere mit dem Studium des Bauortes der Staumauer und mit der Festlegung von Bereich und Umfang der vorläufigen und detaillierten Erkundung. Hier werten wir ausführlich die Methodik der komplexen Erkundungsdokumentation aus. Wir widmen der Bewertung allgemeiner Grundsätze, Grundtypen und Methoden geotechnischer Erkundung große Aufmerksamkeit. Ebenfalls aufgeführt sind die Grundtypen der geotechnischen Feld- und Laborproben. Ein wichtiges Unterkapitel bilden die korrelativen Beziehungen und Bindungen zwischen mechanischen und physikalischen Gesteineigenschaften.

Im letzten Teil beschreiben wir die ingenieurgeologische Erkundung des Staudammflutungsgebietes. Wir behandeln ausführlich die Umformung der Ufer durch Abrasion und Abgleiten; es folgen Erkenntnisse, Lösungsarten und Beobachtungen zusammenhängender geodynamischer Prozesse.

In diesem Buch versuchen wir, einige – mit der Erkundung bei Entwurf und Bau der Staudämme zusammenhängende - Probleme aufzuzeigen und mögliche Lösungen und Arbeitsvorgänge voρζuschlagen. Alle analysierten Probleme sind durch konkrete Praxisbeispiele dokumentiert. Wir glauben, dass diese Zusammenfassung unserer langjährigen Erfahrungen nicht nur zu einem positiven Beitrag für Fachleute, sondern auch zur lesenswerten Lektüre für eine – an diesem Thema interessierte - breitere Öffentlichkeit wird.

Die Autoren

Что найдете в книге



Инженерно-геологическая разведка плотин

или „Что нам преподнесло урок“

Предлагаемая публикация предназначена всем интересующимся проблематикой разведки при проектировке и строительстве плотин. Она может помочь взаимной коммуникации и взаимопониманию между инвесторами и проектировщиками с одной стороны и инженерами-геологами, геофизиками, гидрогеологами, геотехниками и другими специалистами с другой стороны. Следующей группой, к которой мы хотим обратиться, является персонал организаций, обслуживающих плотины. Публикацию можно также использовать в качестве учебного пособия для студентов вузов. Книга является результатом работы и опыта инженера-геолога и геофизика на различных плотинах как в Чешской республике, так и на многих плотинах за границей, где мы работали в качестве исполнителей или консультантов или просто эти стройки посетили.

Книга состоит из введения и заключения и остальных восьми специализированных глав. Во всех особое внимание уделяем практическим примерам, потому что весь объем проблематики можно лучше всего понять на конкретных рассматриваемых заданиях. Мы пытались избежать в книге теоретических разборов, чтобы она принесла прежде всего не правила и нормы, а примеры, которые можно встретить в рамках проектной подготовки и работы сооружения. Нашей целью было прежде всего обратить внимание на факты, пренебрежение которыми ведет к удорожанию строительства плотин или к невозможности ее использования с первоначально запланированной целью.

Первые главы обобщают основные критерии проектировки плотин по отношению к определяющим факторам выбора места и типа дамбы. Оцениваются точки зрения геологические, морфологические, климатические, экологические и остальные факторы, влияющие на выбор места. Далее определяются основные задачи и принципы инженерно-геологических изысканий для плотин. Мы считаем важным разделение разведки на этапы, совпадающие с этапами проектирования, а также эффективность разведки при сохранении требований к безопасности строительства. Большое внимание уделяется стратегии проведения разведки с подробным перечнем отдельных действий. Важной частью является характеристика технического задания и проекта разведочных работ, при которых инвестор, проектировщик или просто заказчик разведки формулирует основные требования, которые необходимо решить при разведке.

Далее описывается определение заданий инженерно-геологической съемки интересующей нас территории гидросооружения и с подробными деталями прежде всего створ плотины. Указан рабочий метод составления целевой инженерно-геологической карты. Описан необходимый объем гидрогеологической разведки (предварительной и подробной). Указаны и возможности контроля правильности и функциональности гидрогеологических скважин с помощью геофизических методов, телекамеры и т.п.

Во второй части книги указаны задачи геофизического измерения на различных этапах разведки. Анализируются возможности отдельных геофизических методов для конкретных проблем. Подчеркивается взаимодействие инженера-геолога и геофизика при определении задач для решения, при их проектировании и анализе. Следует анализ объема прямых разведочных работ. Особое внимание уделяется изучению места, затронутого строительством самой дамбы, и ограничивается размер области и объема разведки для предварительной и подробной разведки. Отдельная подглава подробно рассматривает оценочную методику комплексной документации разведочных сооружений. Большое внимание уделено оценке общих принципов, основных типов и методов геотехнической разведки. Указаны основные типы геотехнических испытаний, как полевых, так и лабораторных, для определения параметров, необходимых для проектирования. Важной подглавой являются корреляционные связи и связи между механическими и физическими свойствами горных пород.

В последней части описывается инженерно-геологические изыскания охранной области плотины. Мы занимаемся прежде всего деформацией берегов водохранилищ под действием абразии и сползания. Предлагаем сведения, способы решения и мониторинга развития связанных с этим геодинамических процессов.

Мы попытались наметить в книге некоторые проблемы, связанные с разведкой при проектировании и строительстве плотин, и предложить возможные решения и рабочие методы. Все главы содержат примеры. Мы верим, что обобщение нашего многолетнего опыта будет полезно не только для специалистов, но и станет интересным чтением для широкого круга читателей, интересующихся данной проблематикой.

Афтори

“Тугонларнинг инженер-геологик разведкаси”

Китобида нималарни топиш мумкин

ёки “Биз учун нима дарс булди”

Ушбу таклиф этилаётган нашр барча тугонларни лойихалаштириш ва куриш ишларида разведка масалалари билан кизикувчилар учун мулжалланган. У бир томондан инвесторлар ва лойихачилар уртасида, иккинчи томондан эса инженер-геологлар, геофизиклар, гидрогеологлар, геотехниклар ва бошқа мутахассислар уртасида узаро муносабатларни урнатиш ва узаро англашувга эришишда ёрдам килади. Биз яна мурожаат этмокчи булган кейинги гурух бу тугонларда хизмат килувчи ташкилотларнинг ходимларидир. Бу нашр, шунингдек, олий укув юртлари талабалари учун укув кулланмаси булиб хизмат килиши мумкин. Китоб инженер-геолог ва геофизик Чех Республикасидаги турли тугонларда хамда чет элдаги куплаб бошқа тугонларда олиб борган ишлари ва тажрибаси натижаси булиб, биз хам у ерда бажарувчилар ёки маслахатчилар сифатида ишлаганмиз ёки бу курилишларни бориб курганмиз.

Китоб кириш ва хулоса кисмларидан хамда саккизта махсус боблардан иборат. Уларнинг хаммасида асосий ахамият амалий мисолларга берилган чунки барча мавжуд масалаларни конкрет олинган топшириклар мисолида яхширок тушуниш мумкин. Китоб аввало коида ва нормаларни эмас, балки лойихани тайёрлаш ва иншоотнинг ишлаши жараёнида учратиш мумкин булган мисолларни акс эттириши учун биз унда назарий бахслардан нарирок булишга интилдик. Максадимиз авваламбор эътиборни шундай фактларга каратиш эдики, уларга эътиборсизлик тугонлар қурилишининг кимматлашиб кетишига ёки улардан дастлаб қўйилган максадларда фойдаланиш мумкин булмай колишига олиб келади.

Биринчи боблар тугонларни лойихалаштиришнинг дамбанинг жойи ва турини танлашга оид хал килувчи омилларга тааллукли асосий мезонларини умумлаштиради. Жой танлашга таъсир этувчи геологик, морфологик, иклимий, экологик мулохазалар ва бошка омиллар бахоланади. Кейин тугонлар буйича инженер-геологик изланишларнинг асосий максад ва принциплари белгиланади. Биз разведкани лойихалаштириш боскичлари билан мос келадиган боскичларга булишни хамда курилиш хавфсизлиги талаблари сакланган холда разведка ишларининг самарадорлигига эришишни мухим деб биламиз. Алохида ишларнинг батафсил руйхатини тузиш билан разведка ишларини олиб бориш стратегияси катта ахамият касб этади. Мухим кисмлардан бири техник топширик ва разведка ишлари лойихасининг характеристикаси булиб, бунда инвестор, лойихачи ёки разведка буюртмачиси разведка ишлари жараёнида бажарилиши лозим булган талабларни ифодалаб беради.

Кейин бизни кизиктирган гидроиншоот худудида олиб бориладиган инженер-геологик съемка топшириклари ифодаланади, айникса тугоннинг урнашиш жойи батафсил ёритилади. Максадли инженер-геологик карта тузишнинг иш услуби курсатилган. Гидрогеологик разведканинг (дастлабки ва батафсил) керакли хажми келтирилган. Геофизик усуллар, телекамера ва бошкалар ёрдамида гидрогеологик бурги кудукларининг тугрилиги ва фаоллиги устидан назорат олиб бориш мумкинлиги курсатилган.

Китобнинг иккинчи кисмида разведканинг турли боскичларида геофизик улчовларнинг максадлари келтирилган. Конкрет масалалар учун алохида геофизик усуллардан фойдаланиш мумкинлиги тахлил килинади. Ечилиши зарур масалаларни аниклашда, уларни лойихалаштириш ва тахлил килишда инженер-геологлар ва геофизикларнинг узаро хамкорлиги таъкидланади. Кейин тугридан-тугри разведка ишларининг хажми тахлил килинади. Асосий эътибор дамба курилиши учун ажратилган жойни урганишга каратилади ва дастлабки ва батафсил разведка утказиш учун разведка худуди ва хажми белгиланади. Алохида булим разведка иншоотларини комплекс хужжатлаштиришни бахолаш услубига багишланган. Геотехник разведканинг умумий принципларини, асосий турлари ва усулларини бахолашга катта ахамият берилган. Лойихалаштириш учун мухим параметрларни аниклаш максадида дала шароитида ва лабораторияда утказиладиган геотехник тажрибаларнинг асосий турлари курсатилган. Алохида булимда тог жинсларининг корреляцион муносабатлари ва уларнинг механик ва физик хоссалари уртасидаги муносабатлар ифодаланган.

Китобнинг охирги кисмида тугоннинг куриклаш зонасида инженер-геологик изланишлар ифодаланган. Биз энг аввало абразия ва сирганиш таъсирида сув омборлари киргокларининг деформацияси билан шугулланамиз. Шу билан боглик геодинамик жараёнларнинг ривожланиши, уларни хал этиш усуллари ва мониторинги тугрисида маълумотларни таклиф этамиз.

Ушбу китобда тугонларни лойихалаштириш ва куришда разведка ишлари билан боглик айрим муаммоларни белгилашга харакат килдик ва мумкин булган ечимлар ва иш усулларини такдим этдик. Барча бобларда мисоллар келтирилган. Ишонамизки, куп йиллик тажрибамизни умумлаштириш нафакат мутахассислар учун, балки берилган масалалар билан кизикувчи кенг омма учун хам фойдали булади.

O knjizi

Inženjersko-geološka istraživanja za brane, odnosno „Šta smo još naučili“

Ova publikacija je namijenjena ljudima koji su zainteresovani za problematiku istraživanja pri projektovanju i izgradnji brana. Knjiga može dobro poslužiti pri zajedničkom radu i postizanju dogovora između investitora i projektanata na jednoj strani i inženjerskih geologa, geofizičara, hidrogeologa, geotehničara i ostalih stručnjaka na drugoj strani. Također želimo zainteresovati lica koja su zaposlena u organizacijama za upravljanje branama. Publikacija može poslužiti i kao udžbenik ili dopunska literatura na fakultetima. Knjiga je rezultat rada i iskustva inženjerskog geologa i geofizičara pri radu na branama u Češkoj Republici i na nizu brana u inostranstvu na kojima su oba autora radili kao odgovorna lica ili konsultanti, ili su samo posjetili ove objekte.

Knjiga sadrži uvod, zaključak i osam stručnih poglavlja. U svim poglavljima naročito potenciramo primjere iz prakse, jer kroz konkretne zadatke i primjere je moguće najbolje shvatiti širinu ove oblasti. Trudili smo se da izbjegnemo teorijske analize i da umjesto normi i teorema ponudimo primjere sa kojima se srećemo u pripremnoj fazi projekta i pri funkcionisanju objekta. Naš cilj je bio da prije svega upozorimo na činjenice, koje ne smiju biti zanemarene ukoliko želimo da izbjegnemo povećanje troškova izgradnje i osiguramo funkciju brane u punom kapacitetu i u skladu sa planiranom namjenom.

Početna poglavlja predstavljaju sažetak osnovnih kriterija pri projektovanju brana u odnosu na ključne faktore za izbor mjesta i tipa brane. Evaluirano je više aspekata: geološki, morfološki, utjecaj klime, ekološki i drugi koji imaju utjecaj prilikom izbora mjesta za izgradnju brane. Definisani su osnovni radni zadaci i principi inženjersko-geološkog istraživanja za brane. Smatramo da je važno podijeliti istraživanja na pojedine etape koje odgovaraju etapama projektovanja i vršiti efektivna istraživanja uz poštovanje zahtjeva za sigurnost tokom izgradnje objekta. Velika pažnja je posvećena strategiji realizacije istraživanja zajedno sa detaljnim spiskom radnih zadataka. Važan dio je i karakteristika projektnog zadatka i programa istražnih radova u kojim investitor, projektant ili jednostavno naručilac formuliše osnovne zadatke, koji bi trebali biti riješeni istraživanjem. Definisan je zadatak inženjersko-geološkog kartiranja interesne oblasti, sa detaljnijim kartiranjem u oblasti mjesta brane. Naveden je proces radova pri sastavljanju inženjersko-geološke karte. Opisan je potrebni raspon hidrogeoloških istraživanja (preliminarnih i detaljnih). Navedene su i mogućnosti kontrole ispravnosti i funkcionalnosti hidrogeoloških bušotina uz pomoć geofizičkih metoda, kamere i sl.

U drugom dijelu knjige su navedeni zadaci geofizičkog mjerenja u raznim etapama istraživanja. Analizirane su mogućnosti aplikacije pojedinih geofizičkih metoda na rješavanju konkretnih problema. Naglasak je stavljen na saradnju inženjerskog geologa i geofizičara pri definisanju zadataka koji trebaju biti riješeni prilikom projektovanja i obrade. Nakon toga slijedi analiza obima direktnih istražnih radova u kojoj se prije svega fokusiramo na istraživanje područja na koje je vršen utjecaj prilikom izgradnje brane i na određivanje područja i obima radova za preliminarna i detaljna istraživanja. Detaljno je analizirana metodika kompleksne dokumentacije istražnih radova. Velika pažnja je posvećena procjeni općih principa, osnovnih tipova i metoda geotehničkog istraživanja. Navedeni su osnovni tipovi geotehničkih in situ i laboratorijskih opita. Jedan od važnijih dijelova knjige je posvećen predstavljanju korelacijskih odnosa između mehaničkih i fizičkih svojstava stijena.

Posljednji dio knjige je posvećen opisu inženjersko-geološkog istraživanja buduće akumulacijske oblasti brane. Uglavnom smo posvetili pažnju izmjenama obala akumulacije utjecajem abrazije i klizanja. Navodimo iskustva, načine rješavanja i monitoringa razvoja odgovarajućih geodinamičkih procesa.

U knjizi smo nastojali da skiciramo neke od problema koji su povezani sa istraživanjem prilikom projektovanja i izgradnje brana i da predložimo moguća rješenja i radne procedure. Svi navedeni problemi su ilustrovani pomoću konkretnih primjera iz prakse. Vjerujemo da će sažetak naših višegodišnjih iskustava biti od koristi ne samo stručnjacima, nego da će predstavljati zanimljivo štivo i za širu javnost koja se interesuje za ovu oblast.

Autori

关于本书
大坝的工程地质调查

——“经验与教训”

本书旨在对大坝设计和建造中的调查工作感兴趣的人们作这方面的阐述。一方面，它为投资者和设计工程师们提供了可以相互了解的平台；另一方面，它也很好的促进了工程地质专家、地球物理专家、水文地质专家、地质技师以及其他专家之间的沟通协调。还有需要我们重视的群体就是大坝的建筑工人。同时本书可以作为学院及大学相关专业教学的教材。本书凝聚了一位工程地质专家和一位地球物理专家在国内不同大坝参与工作以及在国外所访问的多个大坝建设中作为专家和顾问的经验。

本书由引言，结束语和八章技术类正文组成。总体来说我们特别重视实例的分析，因为使用在实践中处理过的真实案例能够让读者在全局上更好的理解相关问题。我们试图给予读者真实的案例，这些案例在设计或建造大坝期间可能都会遇到，同时尽量避免过多的理论分析和设置新的规则和标准。我们的目标主要是指出由于疏忽导致在大坝施工过程中使成本增加或使得工程不能按照原来设计建造的诸多问题。

第一章总结了大坝设计的基本标准，如关于决定选择大坝位置和类型的因素。我们评价了在大坝位置选择过程中有关地质、形态、气候、生态及其它可能对选址造成重大影响的观点。进一步定义了大坝的工程地质调查的基本任务和原则。我们认为重要的是如同维持建筑安全要求一样，将调查与设计阶段同步分为几个阶段，并且更加重视致力于研究包括单个任务的详细列表的调查策略。

本书的重点部分就是其所特有的技术规范和调查工作的设计，使得投资者、设计工程师或者一般员工都能够符合调查执行的规范明确地表达他们的要求。我们进一步叙述和定义了对存在问题的水文工程结构地区，尤其具体到大坝本身所在位置的绘图工

程地质任务的过程。这一部分介绍了可以精确绘制有特殊用途的工程地质图的技术，同时阐述了初步和详细的水文地质调查的必需范围。我们展示了运用地球物理方法、电视摄影机等手段的优势检验水文地质测井的正确性和功能性的可能性。

本书的第二部分介绍了在不同调查阶段地球物理测量的任务。我们分析了用单一的地球物理方法解决具体问题的可能性，我们把重点放在工程地质专家和地球物理专家在定义任务被解决，其中包括设计执行的合作上。通过对既定目标调查工作范围的分析，我们特别注意研究了由大坝自身建筑直接影响到的地方，这决定了初步和详细调查的尺度和范围。

我们详细地评估了调查工作细致复杂的成文方法论，同时很注重对基本原理、准则和进行地质技术性调查的类型及方法的评价。我们引入了包括野外和实验室的地质技术性测试的基本类型为决策获取必要的参数。

岩石的机械和物理性质之间的相互关系也是很重要的一个分章。在最后一部分我们将探讨大坝洪水区的工程地质调查。特别是我们通过改造堤岸的由磨损和滑动造成的水文工程结构来解决此类问题，同时提出了控制和监测与之相关的地球动力学的过程的认识和方法。

总而言之，在本书中，我们试图概述一些关于大坝在设计和修建中调查决策的问题，并且给出可能的解决方案和工作流程。所有章节都以实例说明问题，相信通过我们长期的经验总结不仅会对专业人士提供建议，而且也会对感兴趣的读者有所裨益。

作者
日

حول الكتاب

المسح الجيولوجي الهندسي الخاص بإنشاء السدود
او "الخبرات التي علمتنا"

يخاطب هذا الكتاب المهتمين بالمسح الجيولوجي الهندسي الخاص بتصميم وبناء السدود. ويمكن الاستفادة منه في خدمة التفاهم المتبادل بين الممولين والمستثمرين والمصممين من ناحية وبين المختصين بالجيولوجيا الهندسية والجيوفيزياء والهيدرولوجية والجيوتكنيك وغيرها من ناحية أخرى. كما أردنا ان نخاطب العاملين في المؤسسات التي تستثمر وتشغل السدود.

يمكن كذلك استعمال الكتاب كمنهج للتدريس في المراحل الدراسية في الكليات والمعاهد الخاصة بهندسة السدود.

قد تم في هذا الكتاب تدوين معلومات وخبرات وتجارب مكثفة حصل عليها خبير مختص في مجال الجيولوجيا الهندسية وآخر في مجال الجيوفيزياء أثناء إنشاء سدود مختلفة في الجمهورية التشيكية وفي مناطق أخرى على مستوى المعمورة حيث عمل المؤلفان كخبيرين أو كمستشارين أو فقط قاما بزيارة هذه السدود.

يحتوي الكتاب على مدخل و استنتاج وخلاصة الابحاث و ثمانية فصول. وفي كل فصل من الفصول تم الاعتماد على أدلة وبراهين عملية وخلاصة الابحاث والدراسات التي قد تم تطبيقها في الميدان العملي. اذ أن توضيح الامثلة والنماذج الميدانية تؤدي الى شرح مفصل للأسس العملية لكي يستسيغها الدارس لهذا العلم.

حاولنا قدر الامكان ان نبتعد عن التحاليل والنظريات المملة ولكننا في الوقت ذاته حاولنا بدلا من ذكر القواعد والأسس أن نقدم أمثلة يمكن لقاءها في مجال التخطيط وجريان المراحل المختلفة لإنشاء السدود. اننا نهدف خاصة الى توضيح الوقائع التي تكاد ان تكون ثابتة في تنفيذ المشاريع ومن المفيد جدًا الاعتماد عليها ويمكن عدم أخذها بعين الاعتبار أن يؤدي الى ارتفاع نفقات بناء السدود او عرقلة تشغيلها كما كان يُتوقع من تصميمها.

يتحدث الفصل الاول عن الاسس والنظم الهندسية التي تستخدم في اختيار الموقع الذي يتناسب مع نوعية السد. كما نتحدث عن وجهات النظر الجيولوجية والتضاريس والطوبوغرافيا والمناخ والبيئة والعوامل الاخرى التي يجب اتخاذها بنظر الاعتبار في اختيار الموقع المناسب لإنشاء السدود.

كما تحدثنا عن اهمية المسح الجيولوجي الخاص بإنشاء السدود، ما نعتبره أساسيًا هي توزيع المراحل المختلفة للمسح الجيولوجي الهندسي التي يتم تنفيذها بشكل متناسق مع مراحل تصميم المشروع وحتى نهايته مع المحافظة على متطلبات سلامة العمل. لقد حظيت استراتيجية عملية المسح باهتمام بالغ كونها تحتوي على عدة فعاليات فردية ومهام حساسة في تنفيذ المشاريع.

ويحتوي الكتاب على معلومات قيمة حول المواصفات التقنية الهندسية حول التصميم وأهمية المسح الجيولوجي الهندسي لتنفيذ المشاريع التي يقوم المستثمر أو المهندس أو المصمم وباختصار من يطلب المسح بتحديد متطلباته الأساسية التي يجب الاجابة عليها عن طريق المسح هذا من ناحية ومن ناحية أخرى كي يتم تنفيذ المشروع بالكيفية الضرورية للغرض المنشود له .

كما حاولنا ان نتعرف على كافة النشاطات المتعلقة بالجيولوجيا الهندسية، والمهام المتعلقة، وفقرة الخرائط الجغرافية للمنطقة المعنية والمختارة لإنشاء السد .

لقد تم توضيح الاسس الخاصة بالجيولوجيا الهندسية والهيدرولوجية التي تتعلق بمراحل تنفيذ المشروع (كلتا الحالتين البدائية والمفصلة). وكذلك ذكرنا امكانيات التأكد من صحة ووظيفة الآبار الهيدروجيولوجية باستعمال الطرق الجيوفيزيائية والكاميرات التليفزيونية وما الى ذلك من ادوات خاصة بتنفيذ هذا النوع من المهام.

يتحدث الشطر الثاني من الكتاب عن المهام الجيوفيزيائية وكيفية الاستفادة منها . كما تم شرح بعض الطرق الخاصة بحل العوائق التي تواجه المشروع من ناحية المهام الجيوفيزيائية.

وتم التركيز أيضًا على كيفية وضع آليات وأسس من أجل التنسيق والتعاون في انجاز المهام بين المهندس الجيولوجي والمهندس الجيوفيزيائي.

ومن أجل الاختيار الأفضل للموقع وتحديد نوعية السدود للحصول على أفضل النتائج تم القاء الضوء على كيفية البناء ومراحل الإنشاء، مع أخذ الحجم والأطر والتصاميم البدائية والتفصيلية بعين الاعتبار.

وكذلك تمت دراسة كيفية توضيح وتحليل البيانات في مستندات وملفات البناء ومراحل إنشاء السد. كما قمنا بشرح كيفية تخمين وتقييم المبادئ والقواعد المتبعة والمراجع الاساسية في المسح الجيولوجي في مراحل إنشاء السد.

كذلك قمنا بشرح القواعد الاساسية لانجاز مهام المختبرات والفحوصات الضرورية من اجل تحديد الفقرات الرئيسية للتصاميم وللاعتماد عليها لاحقا في عملية البناء.

كما تمت دراسة العلاقة بين مواصفات الصخور من الناحية الميكانيكية والفيزيائية.

وفي الفصل النهائي تم توضيح مجرى المياه في السدود وكيفية السيطرة على الفيضانات والمكامن الخطرة.

تمت دراسة قضيةِ تَشكيلِ مجرى الضفاف والتيارات المائية وكيفية تنظيمها منوجهة نظر الجيولوجيا الهندسية. كما تم تقديم معلومات حول طرق إتْقان ومراقبة تطور العملياتِ الجيوديناميكيةذات العلاقةِ بمراحل الانشاء.

كما تم القاء الضوء على بعض النقاط الرئيسية التي تتعلق بالمسح الجيولوجي خلال مرحلة التصميم والبناء وكيفية وضع الحلول الرئيسية للمشاكل التي تواجه إجراءات تنفيذ المشروع.

تم شرح كافة النقاط الرئيسية من خلال النماذج والأمثلة الميدانية. ونتمنى بان الجهود التي تم بذلها في جمع المعلومات في طيات هذا الكتاب ستكون مفيدة جدًا، ليس للمختصين في مجال بناء وإنشاء السدود فقط ولكن كي تكون بداية لآفاق جديدة ودراسة علمية وهندسية تفصيلية تكون في متناول يد العامة.

O čem je kniha

Inženýrskogeologický průzkum pro přehrady,

aneb „Co nás také poučilo“

Předkládaná publikace je určena zájemcům o problematiku průzkumu při projektování a výstavbě přehrad. Může dobře sloužit ke vzájemnému pochopení a dorozumění mezi investory a projektanty na jedné straně, na druhé straně mezi inženýrskými geology, geofyziky, hydrogeology, geotechniky a dalšími specialisty. Další skupinou, kterou jsme chtěli oslovit, jsou pracovníci organizací provozujících přehrady. Stejně tak může posloužit jako učební text pro studium na vysokých školách. Kniha je výsledkem prací a zkušeností inženýrského geologa a geofyzika na různých přehradách v České republice a na mnoha přehradách v zahraničí, kde oba autoři pracovali jako řešitelé či konzultanti nebo tyto stavby jen navštívili.

Kniha zahrnuje úvod a závěr a dalších osm odborných kapitol. Ve všech klademe zvláštní důraz na příklady z praxe, neboť nejlépe lze pochopit celou šíři problematiky z konkrétních řešených úkolů. Snažili jsme se, aby se kniha vyhnula teoretickým rozborům a aby spíše než poučky a normy přinášela ukázky, se kterými se lze setkat v rámci projektové přípravy i provozu díla. Naším cílem bylo zejména poukázat na skutečnosti, jejichž zanedbání vede ke zdražení výstavby přehrady, nebo nemožnosti ji provozovat v původně uvažovaném záměru.

První kapitoly shrnují základní kritéria projekce přehrad ve vztahu k určujícím faktorům pro výběr místa a typu hráze. Hodnocena jsou hlediska geologická, morfologická, klimatogenní, ekologická a další vlivy mající význam při výběru místa. Jsou definovány základní úkoly a principy inženýrskogeologického průzkumu pro přehrady. Za důležité považujeme rozdělení průzkumu na etapy shodné s etapami projektování a na efektivitu průzkumu při zachování požadavku na bezpečnost stavby. Velká pozornost je věnována strategii provádění průzkumu s podrobným výčtem jednotlivých úkolů. Důležitou součástí je charakteristika technického zadání a projektu průzkumných prací, kde investor, projektant nebo prostě objednatel průzkumu formulují základní požadavky, které by měly být průzkumem řešeny. Je definován úkol inženýrskogeologického mapování zájmového území vodního díla a v podrobnějším detailu zejména přehradního místa. Je uveden pracovní postup na sestavení účelové inženýrskogeologické mapy. Je popsán potřebný rozsah hydrogeologického průzkumu (předběžného i podrobného). Jsou uvedeny i možnosti kontroly správnosti a funkčnosti hydrogeologických vrtů geofyzikálními metodami, televizní kamerou ap.

Ve druhé části knihy jsou uvedeny úkoly geofyzikálního měření v různých etapách průzkumu. Jsou rozebírány možnosti jednotlivých geofyzikálních metod pro konkrétní problémy. Důraz je kladen na spolupráci inženýrského geologa a geofyzika při definování úkolů k řešení, při jejich projektování a zpracování. Následuje rozbor rozsahu přímých průzkumných prací, v němž se zaměřujeme zejména na studium místa dotčeného výstavbou vlastní hráze a na vymezení oblasti a rozsahu průzkumu pro předběžný a podrobný průzkum. Podrobně je zhodnocena metodika komplexní dokumentace průzkumných děl. Velká pozornost je věnována hodnocení obecných zásad, základních typů a metod geotechnického průzkumu. Jsou uvedeny základní typy geotechnických zkoušek, ať již polních či laboratorních. Důležitou podkapitolou jsou korelační vztahy a vazby mezi mechanickými a fyzikálními vlastnostmi hornin.

V poslední části popisujeme inženýrskogeologický průzkum zátopné oblasti přehrady. Zabýváme se zejména přetvářením břehů vodních nádrží abrazí a sesouváním. Předkládáme poznatky, způsoby řešení a monitorování rozvoje souvisejících geodynamických procesů.

V knize jsme se pokusili nastínit některé problémy spojené s průzkumem při projektování a výstavbě přehrad a navrhnout možná řešení a pracovní postupy. Všechny rozebírané problémy jsou dokumentovány konkrétními příklady z praxe. Věříme, že shrnutí našich mnohaletých zkušeností bude přínosem nejen pro odborníky, ale i zajímavým čtením pro širší veřejnost zajímající se o danou problematiku.

Autoři

O čom je kniha

Inžiniersko-geologický prieskum pre priehrady,

alebo „Čo nás aj poučilo“

Predkladaná publikácia je určená záujemcom o problematiku prieskumu pri projektovaní a výstavbe priehrad. Môže tiež dobre poslúžiť na vzájomné pochopenie a porozumenie projektantov na jednej strane a inžinierskych geológov, geofyzikov, hydrogeológov, geotechnikov a iných na strane druhej. Ďalšou skupinou, ktorú sme chceli osloviť sú pracovníci organizácií zabezpečujúcich prevádzku priehrady. Rovnako tak môže poslúžiť ako učebný text pre štúdium na vysokých školách. Kniha je výsledkom prác a skúseností inžinierskeho geológa a geofyzika na rôznych priehradách v Českej republike a na viacerých priehradách v zahraničí, kde obaja autori pracovali ako riešitelia či konzultanti, alebo tieto stavby len navštívili

Kniha obsahuje úvod a záver a ďalších osem odborných kapitol. Vo všetkých kladieme zvláštny dôraz na príklady z praxe, lebo celá šírka problematiky sa najlepšie dá pochopiť z konkrétnych riešených úloh. Snažili sme sa, aby sa kniha vyhla teoretickým rozborom a aby skôr ako poučky a normy priniesla ukážky, s ktorými je možné sa stretnúť v rámci projektovej prípravy a pri prevádzke diela. Naším cieľom bolo predovšetkým zvýrazniť skutočnosti, ktorých zanedbanie vedie k predraženiu výstavby priehrady, alebo nemožnosti ju prevádzkovať tak, ako sa pôvodne uvažovalo.

Prvé kapitoly sumarizujú základné kritériá projektovania priehrad vo vzťahu k určujúcim faktorom pre výber miesta a typ hrádze. Hodnotené sú hľadiská geologické, morfologické, klimatické, ekologické a ďalšie vplyvy, ktoré majú význam pri výbere umiestnenia priehrady. Definované sú základné úlohy a princípy inžiniersko-geologického prieskumu pre priehrady. Za dôležité považujeme rozdelenie prieskumu na etapy zhodné s etapami projektovania a na efektivitu prieskumu pri zachovaní požiadaviek na bezpečnosť stavby. Veľká pozornosť je venovaná stratégii realizácie prieskumu s podrobným vymenovaním jednotlivých úloh. Dôležitou súčasťou je charakteristika technického zadania a projektu prieskumných prác, pričom investor, projektant, alebo objednávateľ prieskumu formulujú základné požiadavky, ktoré by mali byť prieskumom riešené. Definovaná je úloha inžiniersko-geologického mapovania záujmového územia vodného diela a podrobnejšie popísaná úloha prieskumu na mieste pre priehradu. Uvedený je pracovný postup na zostavenie účelovej inžiniersko-geologickej mapy. Popísaný je potrebný rozsah hydrogeologického prieskumu (predbežný i podrobný). Uvedené sú aj možnosti kontroly správnosti a funkčnosti hydrogeologických vrtov geofyzikálnymi metódami, televíznou kamerou a pod.

V druhej časti knihy sú uvedené úlohy geofyzikálneho merania v rôznych etapách prieskumu. Preberané sú možnosti jednotlivých geofyzikálnych metód pri riešení konkrétnych typov problémov. Pritom je kladený dôraz na spoluprácu inžinierskeho geológa a geofyzika pri definovaní úloh ktoré sa majú riešiť, pri ich projektovaní a spracovaní. Nasleduje rozbor priamych prieskumných prác, v ktorom sa zameriavame hlavne na štúdium miesta dotknutého výstavbou vlastnej hrádze a na vymedzení oblasti a rozsahu prieskumu pre predbežný a podrobný prieskum. Podrobne je zhodnotená metodika komplexnej dokumentácie prieskumných diel. Veľká pozornosť je věnovaná hodnoteniu všeobecných zásad, základných typov a metód geotechnického prieskumu. Uvedené sú základné typy geotechnických skúšok, či už poľných alebo laboratórnych. Dôležitou kapitolou sú korelačné vzťahy a väzby medzi mechanickými a fyzikálnymi vlastnosťami hornín.

V poslednej časti popisujeme inžiniersko-geologický prieskum zátopnej oblasti priehrady. Zvlášť sa zaoberáme pretváraním brehov vodných nádrží abráziou a zosúvaním. Predkladáme poznatky, spôsoby riešenia a monitorovanie rozvoja súvisiacich geodynamických procesov.

V knihe sme sa pokúsili načrtnúť niektoré problémy spojené s prieskumom pri projektovaní a výstavbe priehrad a navrhnúť možné riešenia a pracovné postupy. Všetky rozoberané problémy sú dokumentované konkrétnymi príkladmi z praxe. Veríme, že súhrn našich mnohoročných skúseností bude prínosom nie iba pre odborníkov, ale aj zaujímavým čítaním pre širšiu verejnosť, ktorú zaujíma preberaná problematika.

Autori

1 Introduction

Because of the complexity of their structure, the design and building of dams is a technically demanding process. The relationship between a dam and the underlying rock mass that forms its foundation is the crucial factor that will determine its stability and ultimately the safety of the dam itself and the communities and infrastructure that are situated downstream from it. The factors that link a dam to its foundation are therefore among the most important in the whole field of engineering geology. The failure to understand the crucial nature of this relationship was the cause of some of the most tragic civil disasters during the first half of the last century. In other cases, caution has resulted in overlarge constructions that were both conservative in design and economically costly. The development of survey and design methods in dam engineering has been slower compared with other types of civil constructions, but great advances have been made in the last decades. The rapid development of computer technology has enabled the application of new techniques for mathematical modelling and design and new instruments have also become available for use in geological surveys.

In the past, dam sites were located according to the suitability of the local geology and topographic conditions. The design and construction of dams was largely based on experience acquired from previous projects and the criteria used to determine the heights and types of construction used in dams were often empirically based on this. The geologically and topographically advantageous sites that had been identified were quickly used, and it became necessary to build in more and more challenging geological and topographic situations, where the relationship between the structure of the dam and the rock foundation required increasingly detailed understanding. Due to the increasing demand for water as a natural resource for drinking, agricultural irrigation and industrial purposes, as well as its utilization for the generation of hydroelectric energy, it was also necessary to build higher and higher dams. The growing demand for improvements in the quality of human life had to be balanced against the need to minimize the risk of engineering disasters and negative impacts on the ecology of the environment. These are the main pressures that have forced improvements in the techniques used in the design and construction of dams and, inevitably, in the methodology of engineering-geological and geotechnical surveys that provide the critical information on which the later stages of these projects are based.

Rapid developments in soil and rock mechanics, applied geophysics and survey techniques have taken place during the recent decades. At the same time, analysis of the state of stress in rocks and engineering structures has become possible by applying modern methods of mathematical computation. As a result, it is possible to model the inter-relationship between the engineering construction and the rock mass realistically. The outcome has been to introduce progressively safer methods of design and building techniques. At the same time, methods of monitoring the behaviour of prototype engineering structures have also advanced greatly. Dams have thus become real full-size models, in which the observed behaviour can be compared with original predictions. This has provided extremely valuable data that can be fed back into the design of similar structures intended for analogous natural conditions. Further development of survey and design methods is characterized by the need for continuous reconciliation between the predicted and real behaviour of constructions in a given natural environment. It is only these iterative procedures and the application of the most advanced techniques of computational analysis that can guarantee the safe construction of ever larger and technically more demanding structures in a variety of geological conditions. The technical aspects of the design procedure are also linked to the economic and environmental factors so that costs can be contained within reasonable limits consistent with safety, and so that negative impacts on the environment can be avoided.

Modern computer methods for modelling the interaction of an engineering structure and the rock mass on which it stands require the most accurate possible information about the structure and behaviour of the rock mass, particularly how it will deform under stress, and the strength and permeability of the rocks involved. This task is made more difficult because, in most cases, the rock mass cannot be considered to be homogeneous, isotropic and elastic, even though certain approximations are necessary for the preparation of a mathematical model. Therefore, one of the basic tasks of an engineering geologist is to divide the rock mass into quasi-homogeneous blocks and establish the characteristic geotechnical parameters of each block in order to define the system 'dam – rock mass'. It is only by the application of a wide range of methods of engineering geophysics that this task can be carried out. For this reason, in the book, we place special emphasis on these methods.

The scope and amount of work required for an adequate engineering-geological survey of a dam site is not simple to specify, even though there may be a preliminary concept of the design in relation to what is already known about the geology and topography of the site. To a large extent, the requirements of the survey will be determined by the theoretical knowledge and practical experience of an expert engineering geologist. Although every construction will have a different natural setting and therefore its own specific requirements, it will still be necessary to follow certain standard working procedures. The aim of this book is to explain the procedures required to carry out and evaluate engineering-geological surveys for the main types of dam construction. The recommendations are based on many years of experience acquired in the Czech Republic and abroad. The book is primarily intended for engineering geologists who require expert knowledge of the procedures used in carrying out surveys for dams.

It is becoming more and more evident that the need to ensure adequate supplies of water, essential for the survival of the world's growing population, is creating a significant challenge for the geologists, engineers and planners of the 21st century. Steadily increasing demands for water of good quality have made supplies a major theme of international politics and environmental management. At present, it is estimated that 1 billion people on Earth are without an adequate supply of water. Due to changes in global climate, it is anticipated that, in the near future, patterns of rainfall will become increasingly uneven, both in time and space. Reservoirs, created by damming natural drainage and making artificial lakes, are one of the most practical methods of storing water and regulating the supply. It is therefore certain that dam construction will continue to be one of the most important fields of civil engineering.

2 Basic Criteria of Dam Design

A dam is a complex system within which the individual components interact during operation. For this reason, the design of a dam is conceived as a whole, taking into account this inter-relationship between the individual parts. Every dam design starts with the selection of the most suitable dam site, chosen so that the scope of special foundation work is minimized and the costs of construction are kept as low as possible, taking into account the risk of failure of the structure during building and operation. Such an ideal scenario can sometimes come into conflict with water management, energy, ecological, or other interests, which may then become the main factors that determine the site chosen for constructing a dam and reservoir.

Ultimately, the design of every dam must take account of all the factors involved, but the siting in relation to the natural conditions determined by topography and the underlying geology will always be crucial. Above all, it will be necessary to specify the type of construction and the geometry of the dam (its dimensions and the relative position of the principal components of the structures), the types and volumes of natural construction materials required for the dam and the ancillary works, and the basic technical procedures to be used in construction, i.e. loose fill or concreting, diversion of water during construction, drainage or sealing of construction pits, the means of stabilizing foundations by grouting and related building procedures. An important part of the design process will also be the clarification of conflicts of interest brought about by the project and a study of the impact of the water reservoir on the surrounding environment. By implication, the design plan must also include a detailed budget for the costs of construction and capital investment, and a schedule for carrying out and completing the work.

2.1 Function of Dams

Nowadays, water reservoirs are mostly designed for multi-purpose use. This follows from the need to balance the demand for multiple uses against the limitation on available sites and the supplies of water available. This comprehensive concept of the dam is justified by the need to balance water management against energy demands, domestic and industrial uses, ecological pressures and other factors in an economically efficient way.

The function of a dam is the most important factor determining its capacity and the scale and technical details of all related structures. In particular, it will determine the type of foundation, particularly with regard to permeability. The specifications for a grout curtain will be different where losses of water by seepage are permitted by the design, but criteria for the safety of the dam are met in relation to the hydrostatic pressures acting upward. Certain dams are built simply to protect downstream areas from catastrophic flooding at times of high water and are not designed for permanent impoundment of water. Other dams are built to retain water which then returns to underlying aquifers by infiltration (e.g., Paso Seco in the karst terrain of the Vento Basin).

In pumped storage hydroelectric plants, where there is a rapid fluctuation of the water level, it is of crucial importance to ensure the stability of slopes adjacent to the dam. Necessary measures for the diversion of high water during construction and during the operation of a water-retaining structure, including the requirement to relocate a road along the dam crest, will also have a lesser effect on the selection of the type of dam. Last, but not least, military considerations and protection from terrorist attacks require that, in strategic locations, special attention is given to the use of construction materials and methods that will prevent easy destruction of the dam.

2.2 Selection of a Site for Dam Construction

The choice of a site for dam construction is influenced both by technical and non-technical factors. The principal factors determining the most suitable option for construction will be different in each case and every dam is therefore unique in its own way. There will be differences not only in the geological and topographic settings, but also in the cultural, social and financial circumstances of the country or region concerned. In recent years, the issues concerning the protection of original biotopes have become increasingly important. Regional, national and sometimes even international interests in nature conservation, the protection of mineral resources and the rights of indigenous people are all factors that have to be taken into consideration. This means that when a suitable site for a dam is being chosen, it is necessary to consider not only the physical circumstances and the technical solutions that are available, but also the wide variety of social, economic, political, ecological and legal questions that the proposed dam could raise (Fig. 2.2.1). All these aspects require proper consideration so that basic violations of the natural environment and of national and international law can be avoided.

Fig. 2.2.1 Geologists and engineers carry out a reconnaissance of a potential dam site on the River Eroo (a photo by P. Bláha - 2009)

From the point of view of the engineering-geological survey that must be a prerequisite of all dam construction projects, we are interested particularly in the technical factors that govern the selection of the dam site and water reservoir. These factors are as follows:

- Choice of dam foundation in the given geological and topographic conditions so that the risk of failures or defects is minimized;
- Choice of dam foundation so that the cost of construction will be economic;
- Conditions of permeability at the dam site and in the backwater area of the reservoir that govern the technical options for preventing seepage at economic cost;
- Quantification of the effects of the reservoir on the stability of banks and adjacent slopes;

- Quantification of changes in the immediate environment and in the more remote surroundings;
- Clarification of the potential negative consequence in the area downstream from the reservoir where the discharge regime will be changed because of controlled water management;
- Possibilities for the use of local materials in dam construction;
- Accessibility of the construction site and the possibilities of using the existing road network and, if necessary, constructing additional roads and complementary infrastructure; and
- Comparison of the advantages and disadvantages of the selected dam site with other options for siting the dam taking into consideration all the technical, environmental and economic factors listed above.

It is necessary to emphasise that all decisions regarding the siting of dams based on the technical criteria listed above are taken in the context of the state of knowledge and the policies in force at the time of the proposal. If the project is carried out later, then modifications of the original design will be made that will reflect progress in the procedures used for the engineering-geological survey, and in the techniques used for designing and engineering the construction. Changes can also be prompted by circumstances of a non-technical character. For example, the first edition of the State Water-Management Plan established the policy for water management in the former Czechoslovakia at a time when the transition from isolated, mostly single-purpose hydraulic structures to multi-purpose reservoirs and dams was just gaining ground. The second edition of the Plan was already based on the policy of constructing multi-purpose systems. Since 2 000, planning has been moving towards comprehensive hydraulic designs for whole basins and catchments and, since the Czech Republic became a member state of the European Union, international strategies governing the management and quality of water resources have come into force.

The progressive evolution of a comprehensive strategy for the management of water resources in Europe means that the demands for detailed, high-quality engineering-geological and ecological surveys are also increasing. It is therefore necessary to carry out a comprehensive re-evaluation of surveys and projects implemented in previous decades using the most up-to-date technical standards.

2.3 Factors Determining the Selection of the Site and Type of Dam

When structures for the retention of water are being designed, the proper selection of the dam site and the dam height are crucial but the choice of the most suitable type of dam construction is equally important. Basically, dams are classified according to the type of structure and the construction material used. Thus, there are dams constructed using non-cohesive material (earth-fill and rock-fill embankment dams, combined types of earth/rock-fill dams, and hydraulic fill dams) and dams constructed of cohesive material (masonry and concrete). The latter are divided into three main groups: gravity dams, arch dams, and buttress dams. The main construction material used for these is concrete. In addition to the basic types listed, there are a number of transitional types, e.g. combined earth-fill and concrete gravity dams, in which the concrete structure functions as a channel to divert high water through the dam, and there are also dams based on special construction techniques in which prefabricated elements are used, metal, and chamber dams, and the like. There are other natural options for the classification of dams. For instance, it is common to classify dam structures according to their static action that is the principle by which the water pressure is contained by the dam. In this case it is usual to talk of gravity and arch dams.

To select the most suitable type of dams, the determining factors are particularly the natural conditions of the area of interest, namely:

- Geological and topographic conditions of the dam site and the nature of the foundation;
- Geological and seismological conditions affecting the region chosen for construction of the dam;
- Possibilities for using local construction materials, their quantity, quality and accessibility;
- Climatic conditions; and
- Period of time required for the completion of the dam construction.

In addition to the natural conditions listed above, there is a whole spectrum of other indirect factors which can affect the choice of dam construction. They include, for example, the required function of the dam, whether machinery or only human labour can be used, the potential costs involved in proposed designs and building procedures, etc. These indirect factors determine the degrees of economic, social and political freedom that the country can call upon to carry out development projects. All these factors influence each other, and the definitive solution can only be achieved through close cooperation between a team of experts and the local and regional authorities whose aim should be to balance the safe and economic construction of the dam against the need to preserve, as far as possible, favourable environmental conditions.

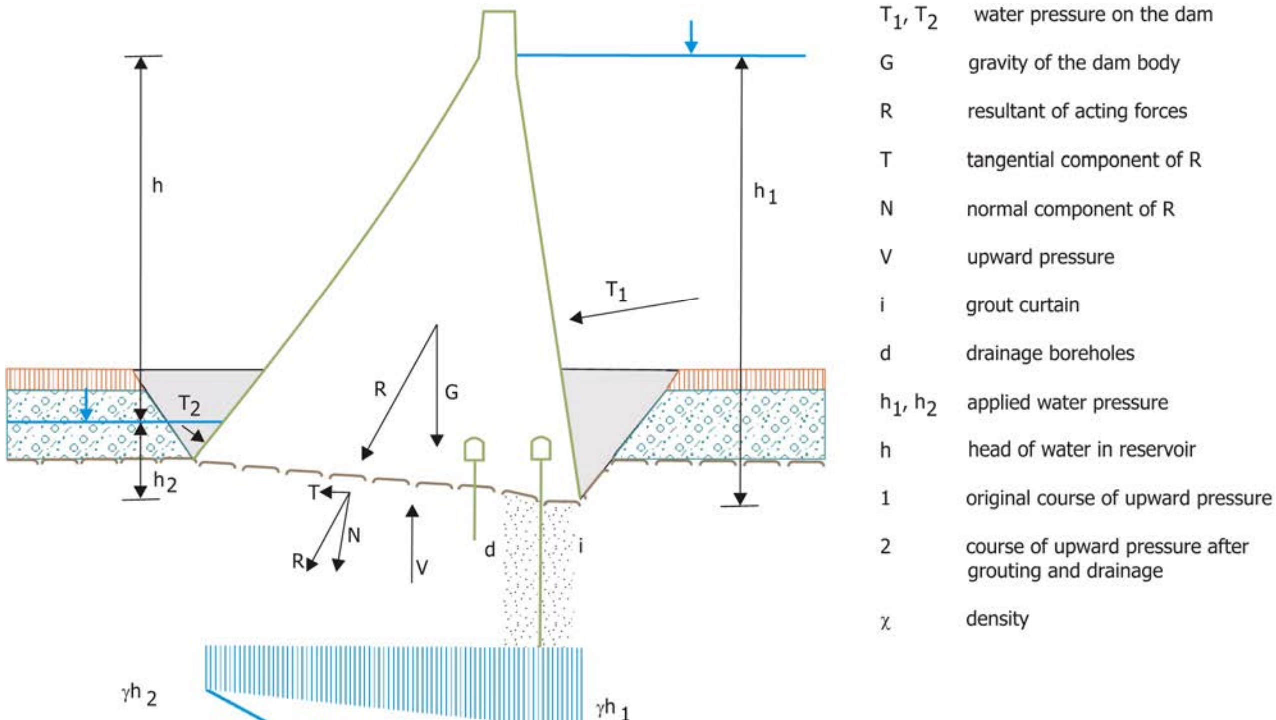


Fig. 2.3.1 Diagram illustrating the transfer of stresses to the bedrock beneath a dam in relation to the hydrostatic pressure exerted on the dam by the contained water

2.3.1 Geological Factors in the Selection of a Dam Site

Geological and tectonic conditions are fundamental factors in selecting a dam site and the type of construction chosen for the dam. They largely determine the safety of the foundation and the costs of construction. All types of dams of any height can be constructed providing the bedrock is strong and the attitude of bedding is favourable and there are no discontinuities that will allow the rocks to move under the stresses and hydraulic pressures created by water and construction of the dam and its foundations. Foundations on solid rock in valleys that are narrow or not too wide are suitable for all types of gravity dams, whereas foundations on rocks and soils with variable compressibility in wide valleys with shallow slopes are best suited to types of earth/rock-fill embankment dams. The geological composition and structure of the site chosen for the dam therefore govern the design of the structure used to contain the water and, to a great extent, determine the techniques that will be used to construct it.

The principle of operation of the concrete gravity dam is that water pressure is transferred onto the rock mass underlying the dam thanks to the resolution of the force of gravity acting both on the mass of water contained behind the dam and the mass of the structure itself (Fig. 2.3.1). The tangential component, T, of the resultant force R must not increase the action of the upward hydrostatic pressure V, because this would operate against the effect of gravity on which the stability of the dam depends. Therefore, the upward pressure is reduced by the grout curtain “i” (line 1) and the drainage boreholes “d” (line 2). The suitability of a site for the construction of a gravity dam therefore depends on the presence of load-bearing rocks on the bottom as well as on the slopes of the valley where the concrete structure is to be placed.

The principal aim of the engineering-geological survey, therefore, is to establish the depth at which sufficiently strong bedrock is present, to determine its mechanical characteristics and to predict the subsidence of individual parts of the construction. In places where the bedrocks are heterogeneous and have different strengths and deform in different ways under stress it is especially important to check that fractures, faults and shear planes do not run in the direction of the main compressive stresses.

When selecting a suitable site for a concrete gravity dam, it is necessary to take into consideration the orientation and dip of the underlying rocks because this will determine the stability of the dam and the permeability underneath the dam. Based on these considerations, a number of typical scenarios for possible foundations are shown in Figure 2.3.2.

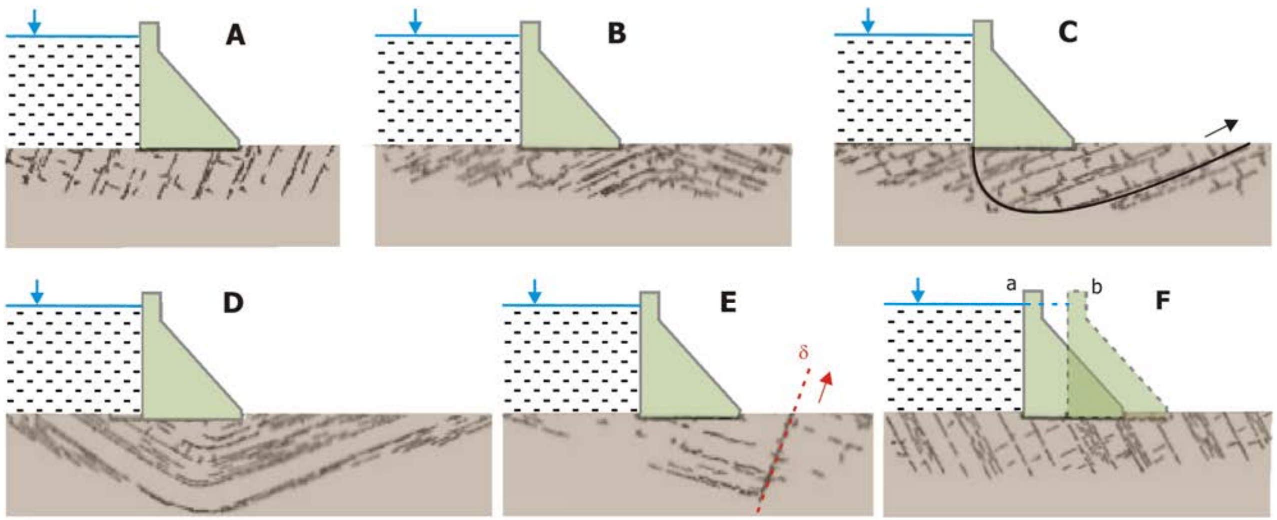


Fig. 2.3.2 Different scenarios for the relationship between a dam and the structure of the underlying bedrock (after Záruba and Mencl, 1957)

In example A, the foundation of the dam rests on favourable bedded rocks striking across a valley and with a steep dip upstream. In example B, the foundation is also stable because the dam rests on the limb of an anticline also dipping upstream against the natural flow of the water. If, however, the bedding dips shallowly upstream against the direction of drainage as shown in examples B and C, the dam can slip downstream along the bedding planes. Example D shows an unsuitable site in which the dam rests on the limb of a syncline dipping downstream. In example E, there is an unsuitable combination of beds dipping downstream with an associated system of fractures that would be the cause of permeability beneath the dam and further downstream. In example F (a), the upstream side of the dam rests on permeable rocks, which will result in increased upward pressures beneath the dam. If the dam is sited so that its upstream face can rest on impermeable beds, as in example F (b), the detrimental effect of upward pressure will be reduced.

Foundations for concrete gravity dams should rest primarily on strong igneous, metamorphic and sedimentary rocks. These types of rocks, if not weakened by joints, faulting or weathering, are usually firmer than concrete and their bearing capacity is usually adequate.

Special attention is required if dams are sited on granitic rocks. Granite is frequently hosts for hydrothermal mineralization located along zones of crushing and alteration and, in certain cases, wide belts of decomposed granite occur in which the feldspars have been altered to clay by hydrothermal action and weathering. The intersection of several sets of fractures in the rock mass along which hydrothermal alteration has occurred will seriously affect the mechanical stability of an otherwise strong rock. In the case of the Liberec Granite Massif in the Czech Republic, several systems of tectonic fractures were detected in the valley bottom during an engineering-geological survey for a dam carried out by Josefův Důl (Fig. 2.3.3).

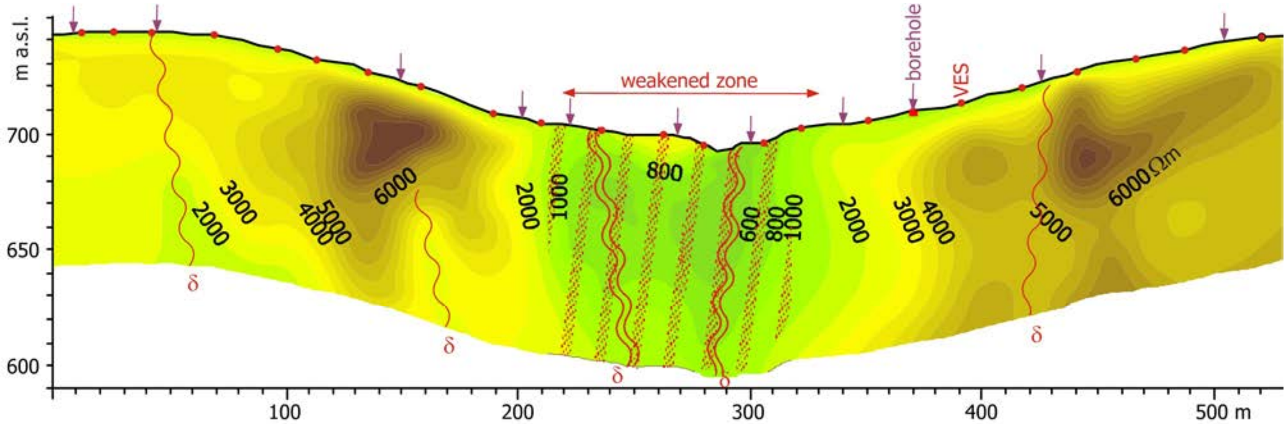


Fig. 2.3.3 Cross section of the site of the dam at Josefův Důl showing the zone of weakened rock beneath the floor of the valley

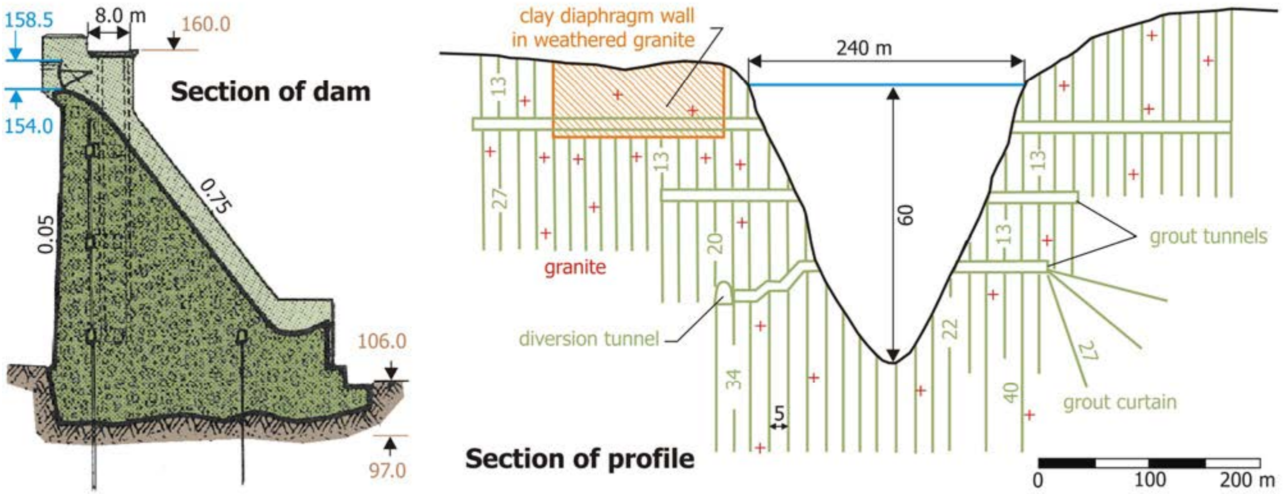


Fig. 2.3.4 The Boadella Dam - an example of a concrete gravity dam (adapted from "Inventario de Presas Españolas" ICOLD, 1973)
left: Cross section of the dam
right: Cross section showing the position of the grout tunnels and the extent of the grout curtain and the clay wall used to prevent leakage

The first geophysical survey was carried out there using only resistivity methods. Devices capable of more extensive seismic measurements, suitable for identifying fracturing in the rock mass, were not available at that time. Subsequent geophysical surveys combined with drilling work revealed a belt of tectonic disturbance in the valley over 100 metres wide, along which the granite was weathered to clayey sand down to a depth of 16 to 17 metres. Relatively undisturbed granite was not encountered until a depth of about 40 metres. Based on these findings, the technical solution adopted was to install a diaphragm cut-off wall in uncased trenches of 17 metres in depth, combined with a grout curtain down to a depth of 40 metres.

A similar situation was encountered during a geological survey for the Boadella dam on the River Muga in the northeastern Pyrenees (Fig. 2.3.4). The dam there is located on fine- to medium-grained leucocratic granite, which is weathered throughout to a depth of about five metres. At the place on the right bank where the dam was keyed, the granite had been decomposed to sand to a depth of 11 to 15 metres due to faulting and hydrothermal alteration. Water pressure tests showed that the altered rock mass was highly permeable. In order to prevent the escape of water by seepage it was necessary to install an impermeable diaphragm cut-off wall. Grouting tests had already shown that this sandy material could not be stabilized by any method of grouting.

The diaphragm cut-off wall was constructed by drilling a series of mutually overlapping boreholes of 600 mm in diameter (Fig. 2.3.5) using a large-diameter percussion drill rig. The holes were drilled to a depth of 18 metres and subsequently filled from the bottom up with compacted clay. At a distance of 2.5 metres behind the diaphragm cut-off wall, and at a depth of 13 to 15 metres, a grout tunnel with two metres clearance was dug. Routine grouting of the granite to a depth of 27 metres was carried out from this tunnel. A view of the dam during construction is shown in Fig. 2.3.6.

Fig. 2.3.5 Diagram showing the position of the clay wall in the trench excavated in weathered granite relative to the fence of holes drilled to enable the injection of grout

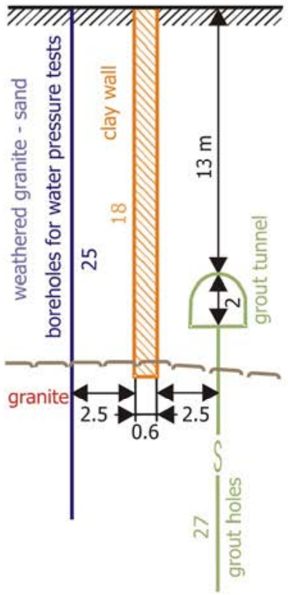


Fig. 2.3.6 View of the Boadella dam under construction (a photo by O. Horský - 1967)

Similar difficulties have also been encountered in apparently strong metamorphic rocks. During an engineering-geological survey for the dam at Dalešice on the River Jihlava, it was found that the metamorphic rocks (amphibolite and granulite) had been pervasively weakened by several intersecting systems of fractures. Zones of mylonite and cataclasite reaching several metres in thickness were formed during several stages of deformation and recrystallization. Intensive fracturing of the rocks was found down to depths exceeding 30–50 metres. For these reasons it

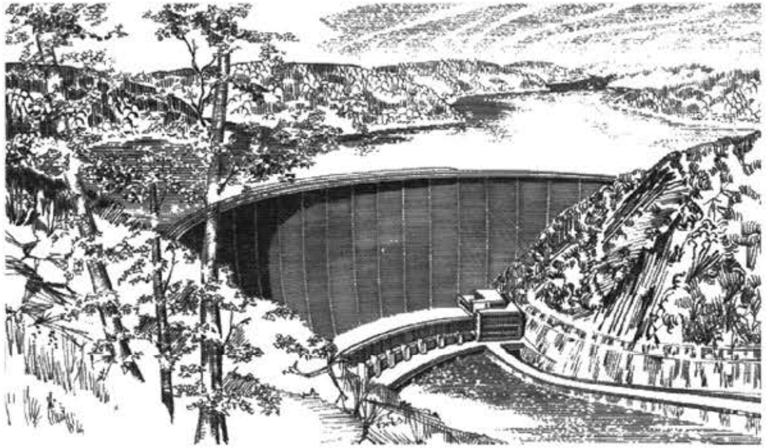


Fig. 2.3.7 Scenario for the construction of an arch dam at Dalešice (after Konečný in Hanák and Mejzlík, 1972)

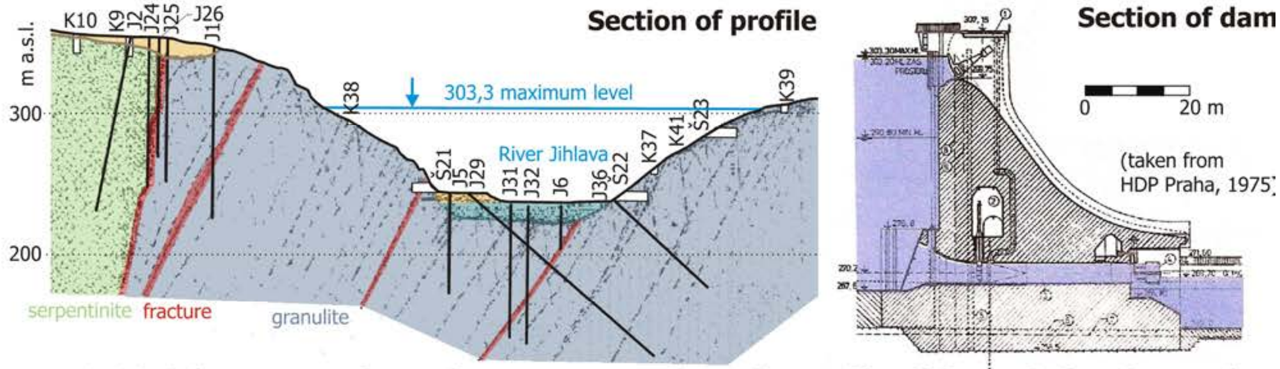


Fig. 2.3.8 Schematic geological cross section along the profile of the Mohelno dam with a cross-section showing the structure of the concrete gravity dam and the penstock to channel water through the HEP turbines (after Krčma, 1975)

turned out to be safer and technically more appropriate to construct a rock-fill dam, even though the construction of an arch dam (Fig. 2.3.7) had originally been considered. The construction of the rock-fill dam was not achieved without technical problems either (Fig. 2.3.29), even though the dam itself was built, the excavation of aggregate for the rock dyke created a number of difficulties.

Problems caused by fracturing of granulite and serpentine bedrocks were also encountered during an engineering-geological survey for the dam at Mohelno (Fig. 2.3.8) on the River Jihlava. This dam had been designed as a balancing reservoir for the Dalešice pumped storage hydroelectric plant (PSHEP). Fracturing of the granulite was not so marked at the site of the balancing dam as in the case of the Dalešice dam so it was possible to construct a concrete gravity dam safely and economically (Fig. 2.3.9).

An example of an unsuitable design for a concrete gravity dam is the Tous dam on the River Jucar. At this dam site, engineering-geological work had been carried out intermittently since 1933. At the time when the excavation at the beginning of construction was being carried out in 1960, it was evident that there were two significant lines of tectonic weakness that cut through the channel in the Cenomanian limestone. They had caused intense crushing of the rock. The width of the main fracture is greater than 50 metres and the zone of fracturing has been followed to a depth of more than 150 metres. This main tectonic fracture runs perpendicular to the line of the dam, i.e. roughly parallel to the general direction of flow, and, in all likelihood was responsible for governing the course taken by the River Jucar. A minor fracture higher in the slope is at an angle of 45° with the dam axis.

Fig. 2.3.9 View of the Mohelno concrete gravity dam (a photo by O. Horský - 1974)

Crushed rock and later Tertiary sediments incorporated in the zone of weakness fill a depression in the valley following this zone to a depth of over 50 metres. The filling of this fracture zone has a clayey character and was the main cause of foundation problems. The unsuitable mechanical properties of these sediments prevented the completion of the already partly constructed concrete gravity dam because of the instability of the foundations. As a result, it was not until 1978, after many years of discussions, that it was possible to proceed with the construction of a 127 metre high gravity dam combined with a rock-fill dam. The already half-built concrete gravity blocks (Fig. 2.3.10) were used to advantage in the later construction work. The overall width of the dam after its completion was 785 metres at the dam crest, the maximum volume of water contained behind the dam reached 80 million m^3. The dam, however, did not operate for long. Shortly after its completion, in October 1982, during a tremendous rainstorm, when 600 mm of precipitation fell in that area within 24 hours, the dam was broken and destroyed. This disaster is described as the most horrible accident of its kind in the history of Spain. A total of 45 Mm^3 of water was released abruptly into the valley of the River Jucar downstream from the dam and caused catastrophic damage and many casualties.

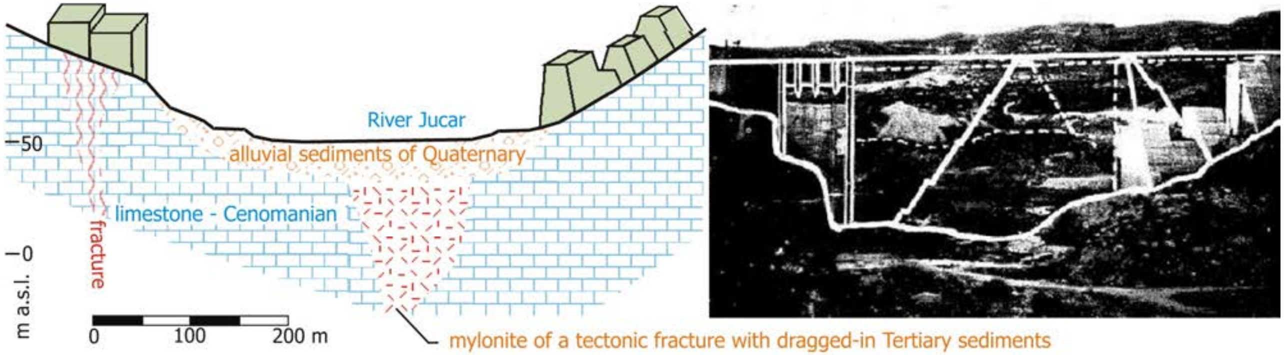


Fig. 2.3.10 Schematic cross section of the valley of the River Jucar, showing the zones of faulting and crushing in the limestone that led to difficulties in the construction of the Tous dam (adapted from "Inventario de Presas Españolas" ICOLD, 1973)

In 1989, the construction of a new earth-fill dam with a central core of clay was begun at Tous, this time an earth-fill dam with a central core of clay. Due to the rising demand for water, the dimensions were enlarged to provide a storage capacity of 340.4 million m^3. The height of the dam was designed to be 135.5 metres and the length of the crest of the dam was 1,024 metres at maximum backwater level. This dam was completed in 1994 and its operation has been trouble-free. At present, after more than ten years of operation, minor repairs will be made and certain ancillary works will be finished (Fig. 2.3.11). The example of the Tous dam illustrates the necessity for a thorough engineering-geological survey and responsible interpretation before decisions are taken on the design for constructing a dam.

Fig. 2.3.11 "Aerial" view of the Tous embankment dam

During an engineering-geological survey for the "El Bosque" dam (Fig. 2.3.12), two deep valleys buried beneath Tertiary volcanic sediments were discovered on the left bank of the designated dam profile. Due to the high permeability of these sediments, the costs of construction of the dam increased dramatically because of the scale of the grouting required to stabilize the foundations.

Sedimentary rocks, such as sandstone, claystone, arkose, greywacke, limestone, and dolomite can provide the suitable subsoil for a concrete gravity dam under certain conditions. The strength of these sedimentary rocks depends on the degree of their lithification and the composition of their cement. Sedimentary rocks with carbonate and clay cements are generally less suitable for gravity dams, and often prove completely unsuitable. Limestone provides good foundations for a gravity dam in certain circumstances but they are susceptible to karstification. Karst can be a cause of major leakage from a dam and therefore, in such circumstances, the site of the dam and the backwater area must be surveyed in detail. It was for these reasons that the dam profile for the downstream reservoir of the Centro Cuba pumped storage hydroelectric plant was abandoned (Fig. 2.3.13).

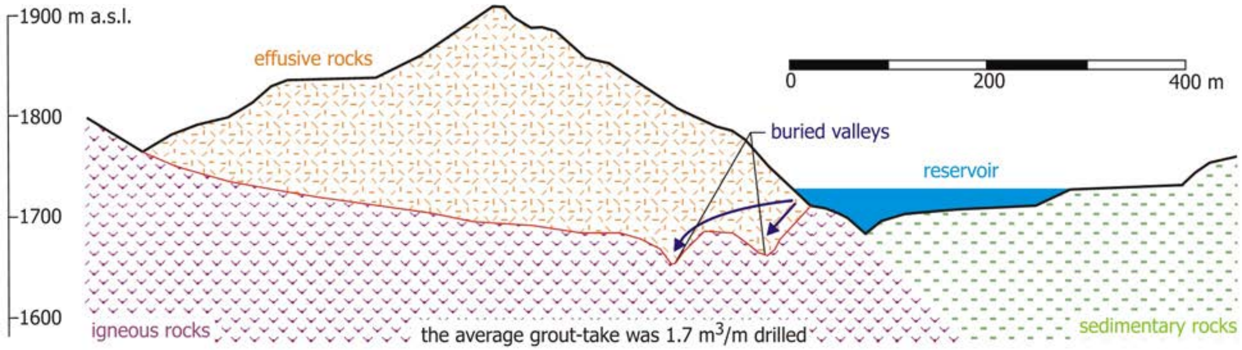


Fig. 2.3.12 Schematic section across the site of the El Bosque dam showing the position of the buried valleys along which leakage of water took place (adapted from "Proceedings of the International Congress on Large Dams", Mexico, 1976)

Weak rocks containing a high proportion of clay minerals are utterly unsuitable subsoils for the concrete gravity dams. These rocks are of little shearing strength, are highly compressible, locally permeable, and susceptible to piping or liquefaction.

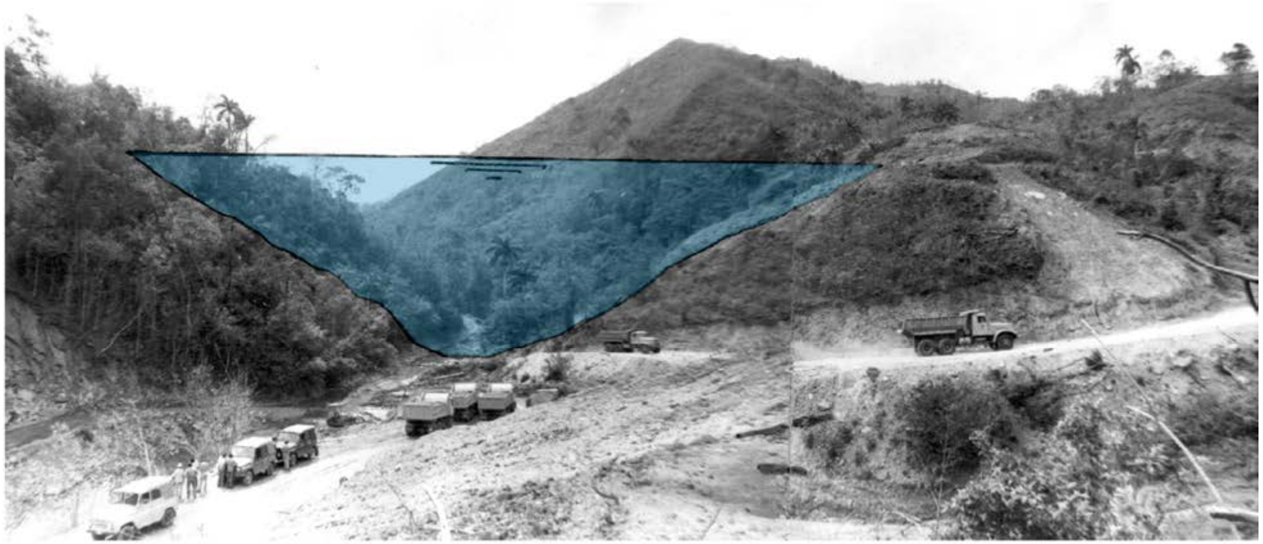


Fig. 2.3.13 Profile deemed unsuitable for the construction of the lower dam of the Centro Cuba PSHEP (a photo by O. Horský - 1986)

On subsoils of low quality, buttress dams with a static action similar to massive gravity dams can be constructed. Moreover, this type of dam construction has a number of advantages. Compared with gravity dams they require less concrete and the dimensions of individual buttresses and the spacing between them can be adjusted to suit the geological conditions so that the loading in the footing beneath the buttresses does not exceed the permissible limit. Due to the independent behaviour of individual buttresses and the choice of spacing between them, tectonically problematic sections can be managed with relative ease (e.g. fault zones). This means that the engineering geologist responsible must identify sites where conditions provide good foundations under the individual buttresses of the dam. The pressures will be concentrated onto the buttresses and it is therefore necessary to ensure that the foundations can effectively resist the shear stresses that will be imposed by the dam (Fig. 2.3.14). Differences in the deformation of individual buttresses must not exceed values set by the design of the dam so that the structural integrity linked through to all the buttresses is not compromised. Lower buttress dams constructed on rocks of low quality can be founded on a concrete base plate which distributes the loading evenly across the foundation level. Difficult sections can be spanned either by a concrete superstructure or by recessing and partial concreting of the fractured rock.

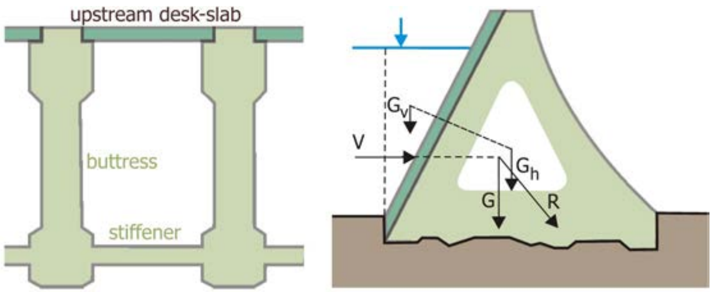


Fig. 2.3.14 Diagram illustrating how the forces exerted by the contained water in a multiple slab buttress dam are transferred to the bedrock. G_V – vertical component of hydrostatic stress; V – water pressure acting on dam; G_h – weight of dam; R – resultant force (adapted after Kos, Zajíc, 1961)

The loading of buttress dams is determined by the same factors that control the loading of gravity dams. In this case, water pressure is more important than in other types of dam, because the vertical component acting on the upstream face of the dam contributes substantially to the stability of the structure. The reinforced-concrete plate at the front places only a small load on the basement, but the

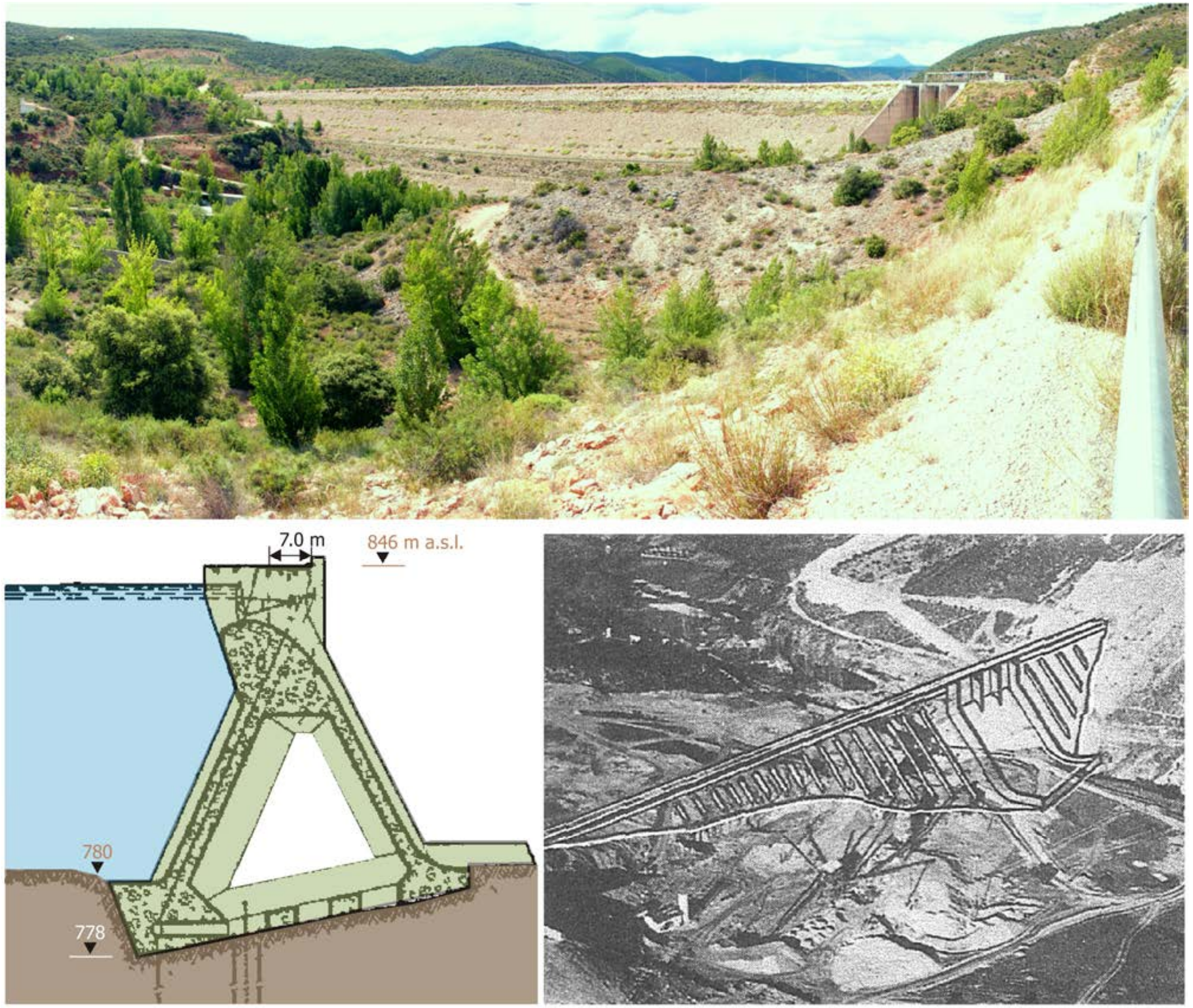


Fig. 2.3.15 Cross section showing the structure of the Beleňa multiple slab buttress dam with a conceptual view of the position of the dam on the chosen profile on the River Sorbe (adapted from "Inventario de Presas Españolas" ICOLD, 1973, a photo by P. Bláha - 2009)

loading under the buttresses is usually greater than in gravity dams. The adverse effects of upward pressure are lower in buttress dams, because the area of the base of the footing is lower as well. Buttress dams are also more suitable than single massive gravity dams for sites in wider valleys because of the smaller quantity of concrete required for construction. In the most advanced designs of buttress dams, the saving of concrete reaches 40 to 60 %. An important task during an engineering-geological survey for a buttress dam is the assessment of the stability of the adjacent slopes. This is because there is a danger that fractured and unstable slopes will transfer pressure onto the dam in a longitudinal direction. This could cause the dam to breach on its flanks. The abutments of lightweight buttress dams are therefore usually constructed as massive blocks. An example of such a method of construction is the Beleňa dam on the River Sorbe (Fig. 2.3.15). For economic reasons, it was later built in another dam profile in the option of a rock-fill dam.

In buttress dams it is necessary to make more allowance for the effects of waves, ice and wind on the upstream face. The dimensions and thickness of the upstream slabs or multiple arches depend on these considerations. The thinner elements of buttress dams are affected directly by the ambient conditions, so temperature changes are very important. As in the case of gravity dams, the decisive factor determining the stability of buttress dams is the possibility of shearing on the foot of the foundation.

In contrast to dams of the gravity type in which the stresses imposed by the mass of water behind the dam add to the stability of the structure, in the case of arch dams the forces are transferred into the foundations on the slopes and subsoil. This places exceptionally high demands on the strength of the rock mass. As a result, the engineering geologist is faced with an extraordinarily challenging task that involves detailed study of all the planes and zones of mechanical weakness in the rock mass. This will include faulting and fracturing of tectonic origin, patterns of jointing, bedding planes, metamorphic foliation and cleavage and any superimposed zones of weathering and/or hydrothermal alteration. In addition to identifying their geological characteristics, it is equally important to determine their geotechnical properties, frequency and spatial orientation in relation to the forces imposed by the intended dam and the water behind it (Fig. 2.3.16).

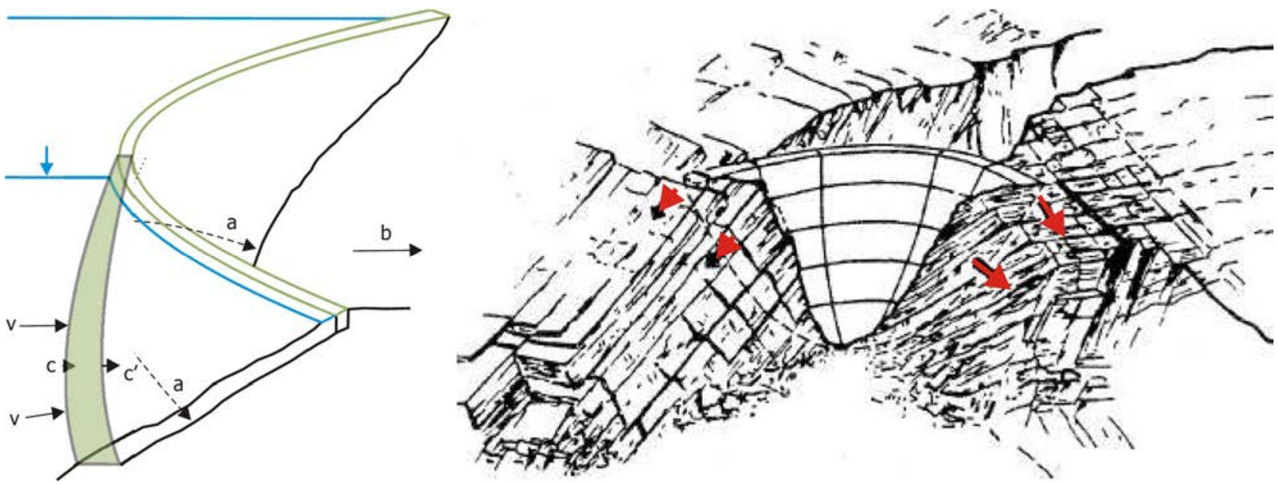


Fig. 2.3.16 Diagrams illustrating the principles of construction of a concrete arch dam and the way in which stress is transferred through the abutments of the arch to the rocks on either side
a – curvature of the arch with convexity facing upstream
b – downstream direction
c – direction of cantilever action on the central profile of the arch
c' – change in the shape of the vertical profile due to the stresses imposed by filling the dam
(after Záruba and Mencl, 1974 and Kos and Zajíc, 1961)

If the planes of mechanical discontinuity are oriented so that the resultant of the acting forces compresses, consolidates, and closes them, it is relatively easy to evaluate their behaviour under the deforming stresses and to allow for this in the calculations. In contrast, if the orientation of the planes of mechanical discontinuity lies close to or in the direction of acting forces, it is essential that the engineering-geological survey takes into account the shear strengths of all the planes of weakness. At the dam site near Dalešice, for example, the unfavourable orientation of the main systems of faults and fractures meant that the construction of an arch dam on the site was not recommended (Horský, Novosad, Müller, 1972, Horský, Müller, 1974, Fig. 2.3.17).

It can be stated as a rule that only solid unweathered rocks fulfil the strict criteria for shear strength, compressibility and impermeability required for constructing the foundations of large concrete arch dams. The geomorphology of the dam site should also be favourable and sites with structural and tectonic complications should be avoided (Fig. 2.3.18).

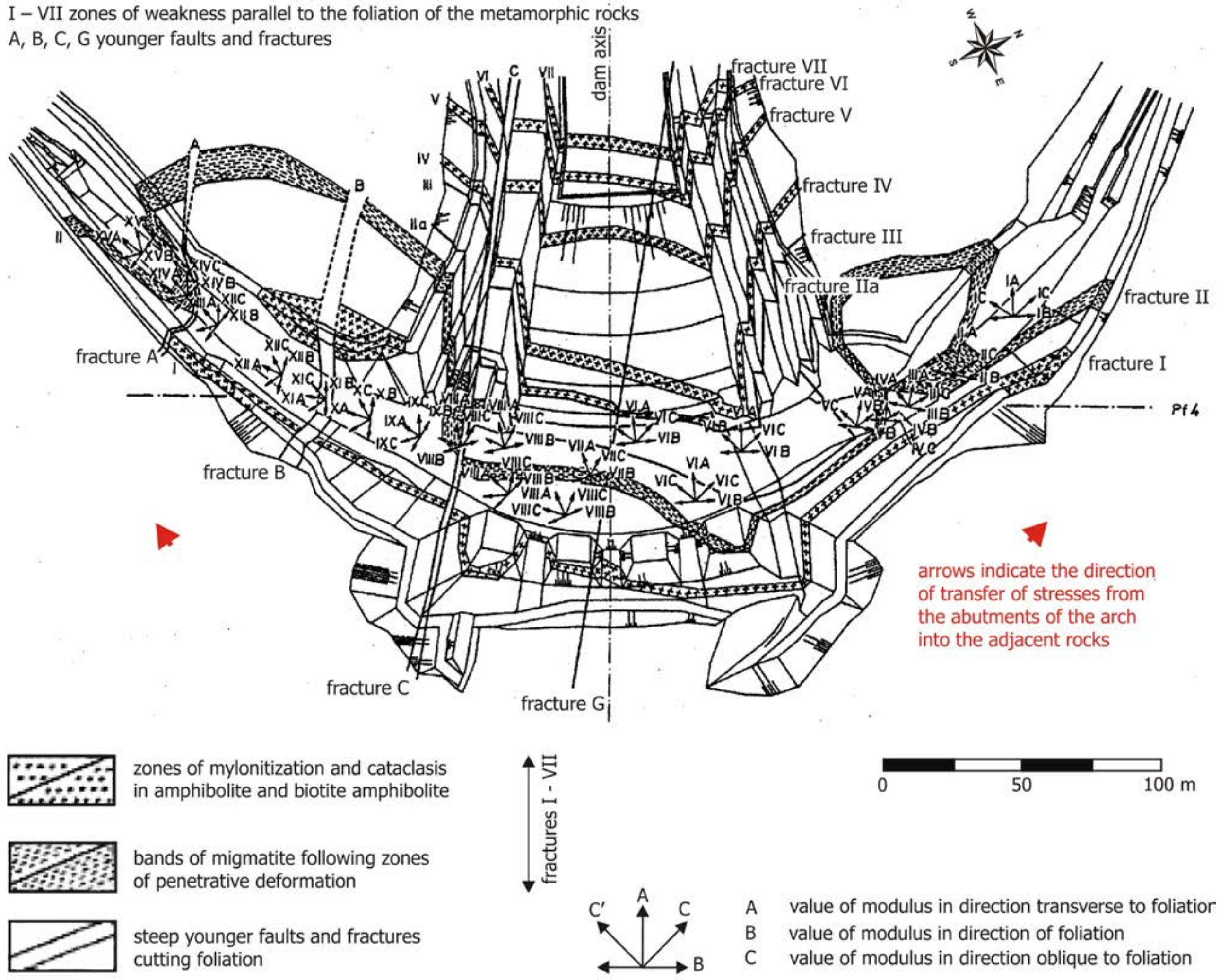


Fig. 2.3.17 Block diagram showing the zones of weakness governing the geotechnical conditions in the proposed excavation for the construction of a concrete Dalešice arch dam

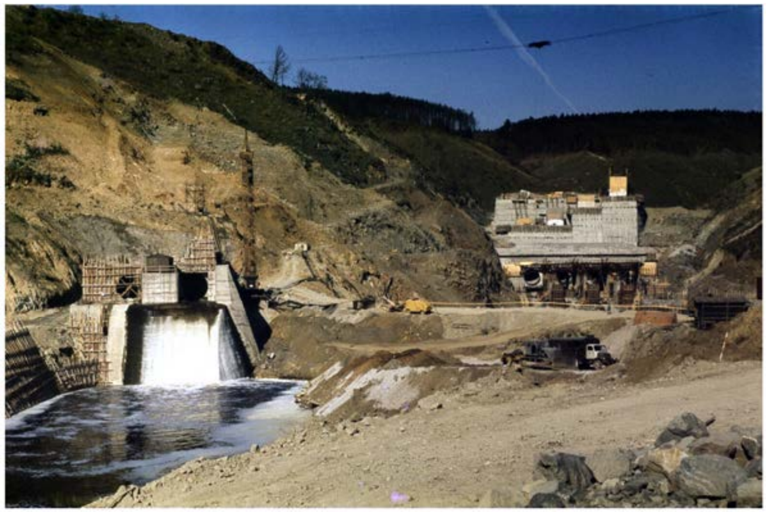


Fig. 2.3.18 View showing the combined concrete and rock fill gravity dam at Dalešice under construction (a photo by O. Horský - 1978)

The Quentar dam site on the River Aguas Blancas is located on dolomite and limestone in the lower part of a valley with nearly vertical walls. In this case the rocks have been affected by tectonics to such a degree that even bedding planes cannot be discerned (Fig. 2.3.19). A site with very favourable morphology was selected for the construction of this arch dam which was successful despite the need to undertake extensive reinforcement of the fractured rocks. In the case of the Susqueda dam in the north-eastern Pyrenees, intense fracturing of the gneiss and diorite, especially in the valley, was identified during the preparatory engineering-geological survey. To protect the concrete arch dam against potential shearing, a number of 30-metre high gravitational step (buttress) supports were constructed (Figs. 2.3.20 and 2.3.21).

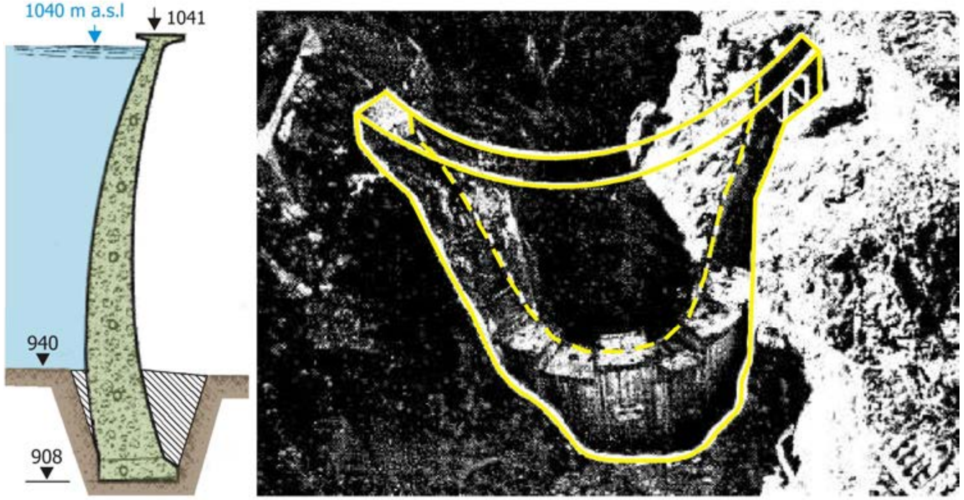


Fig. 2.3.19 Cross section showing the central profile and foundation of the Quentar concrete arch dam with a view of the dam under construction showing the outline of the completed dam projected in yellow colour (adapted from "Inventario de Presas Españolas" ICOLD, 1973)

Sedimentary rocks, especially flysch sequences consisting of alternating beds of claystone and sandstone are generally unsuitable for the construction of arch and buttress dam foundations because of the abundant planes of mechanical discontinuity.

If rocks are easily compressible or show great variations in deformability, they will not provide suitable foundations even for the classical concrete gravity dam. Semi-solid rocks and soils are unsuitable for concrete dams because they do not provide sufficiently strong foundations and their mechanical behaviour is also very sensitive to the effects of water. In such cases it is essential to undertake an engineering-geological survey appropriate for an embankment dam constructed using non-cohesive material.

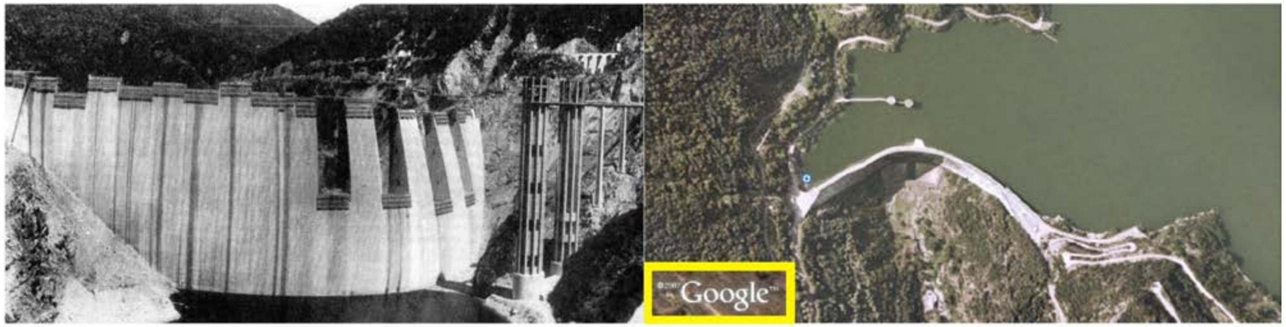


Fig. 2.3.20 View of the Susqueda arch dam under construction in 1966. The Susqueda dam is a three centred, double curvature arch with a height of 135 m above foundations and a crest length of 510 m. The satellite image from Google Earth shows the completed dam with the reservoir filling the Ter valley behind (a photo by O. Horský - 1966)

An engineering-geological survey for the construction of an embankment dam is based on similar principles to one made for the purpose of constructing a gravity dam. However, because the type of construction is different, the relative importance of individual tasks and their objectives are changed. The most important task is to identify a source of suitable construction materials for the construction of the embankment dam in the immediate vicinity of the dam site. These materials must be volume-stable and of the necessary quality to guarantee the stability and impermeability of the dam. If such materials cannot be found within a suitable distance, it is necessary to proceed to another technical solution. For example, in the Valmayor dam on the River Aulencia, an asphaltic concrete layer was applied on the upstream side of the dam with great success (Fig. 2.3.22). In case of the Slezská Harta dam in the Jeseníky Mountains in northern Moravia, construction materials were provided from local resources for both the shoulder and the sealing components (Fig. 2.3.23).

Fig. 2.3.21 Schematic cross section of the Susqueda arch dam (after Rebollo, 1967) together with a photograph showing the step supports at the foot of the arch (a photo by O. Horský - 1996)

A question of extraordinary importance is to determine permeability of the rocks beneath dams and to take appropriate technical measures to reduce the permeability and counteract the harmful effects of water seepage and upward-acting hydrostatic pressures. Grout curtains and specially designed systems of drainage are used for this purpose. Planes and zones of weakness in the geological structure (faults, fracture systems, foliation, bedding planes and zones of hydrothermal alteration), the permeability of the rocks, and their strength and compressibility are the fundamental parameters that govern the safety and efficiency of any type of dam construction.

The preliminary survey must also provide the data necessary to calculate the amount of settlement and compression of the subsoils under the dam. The strength of the rocks in the foundations will determine the stability of the dam. If an embankment dam is built on compressible soils or

Fig. 2.3.22 Oblique satellite view looking west to the Valmayor rockfill embankment dam (Google Earth). Inset, lower left, is a schematic cross section of the dam. Inset, upper left, is a view of the dam under construction in 1978 (a photo by O. Horský - 1978)

soils susceptible to loss of strength in the case of seismic events, an essential part of the engineering-geological survey will be to assess the measures required to consolidate such soils, e.g., reinforcement by grouting or by other methods. An equally important question will also be whether suitable construction materials can be obtained within an economically accessible distance.

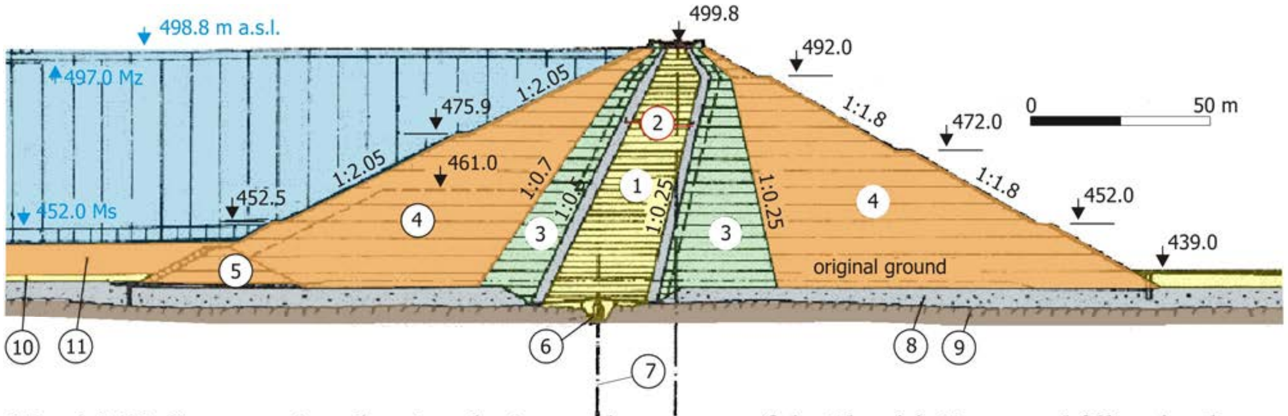


Fig. 2.3.23 Cross section showing the internal structure of the Slezská Harta rockfill embankment dam. The dam rises to a height of 64.8m above the valley floor. 1) central core of local clay; 2) filter layer of imported sand; 3) transitional layer of local gravel and sand; 4) locally quarried basalt forming the main body of the dam; 5) upstream reservoir; 6) inspection and grout tunnel; 7) grout curtain; 8) floodplain gravel; 9) Culm shale; 10) alluvial loam; 11) additional filler (after Janda, 1973)

2.3.2 Topographic Factors in the Selection of a Dam Site

The topographic features along a valley and across a valley have an important influence on the site chosen for a dam and the type of construction that will be used. At the same time it will be necessary to balance the minimum demands for the volume of the dam against considerations of the safety of the type of construction chosen and the way in which this will fit into the natural environment. The suitability of the proposed type of a dam will also depend on the practicalities involved in its construction.

The survey of the morphology of the proposed dam site should accurately determine the shape of the valley along longitudinal and transverse profiles, measure the angles of slopes and their stabilities and ultimately establish the factors responsible for its formation. This information enables the selection of the optimum site for the dam and the methods that will be used in its construction. When selecting a dam site, it is usual to focus on the narrow parts of the valley where the rock slopes are steepest. Such sites offer the most suitable geological conditions. However, this general rule is not always applicable because advantageous narrowing of a valley may have been caused by landslides or rock falls from one or both flanks of the valley. A historical example of this situation is the original site chosen for the Orlík dam (Fig. 2.3.24). It is notable that it was not an engineering geologist, but a classically trained geologist who discovered evidence of an extensive slope failure by careful documentation of field workings.

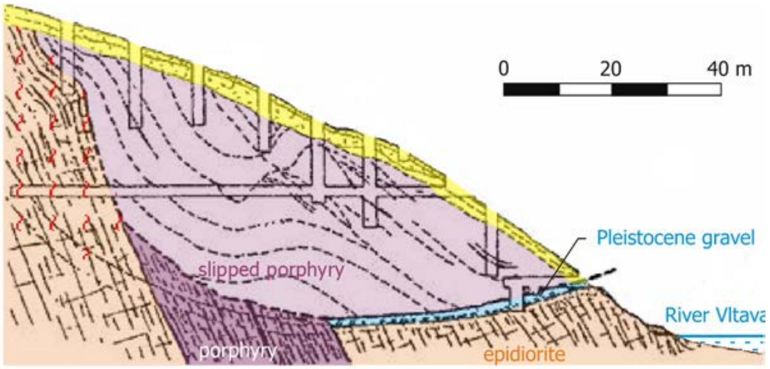


Fig. 2.3.24 Cross section along the profile originally chosen for the Orlik dam showing the rotational slope failure above the Vltava and the exploratory workings excavated in the mass of slipped porphyry

Another example of the siting of a dam in a narrow valley is the Genal dam. During a preliminary survey four options for a dam profile were investigated. Based on the first assessment, Profile P3 seemed to be the most suitable. In this case the ratio of dam height to the width of the dam crest (parameter α) was far the most advantageous. Conventional engineering-geological mapping was made very difficult by dense bushy vegetation that prevented access to the ground, except along narrow paths. Geophysical measurements were also carried out along these paths. The geophysical results clearly indicated that there had been a major slope failure on the flanks of profile P3. The rocks in which the extensive failure had occurred are fractured schist and phyllite, belonging to the complex of the Betic Cordillera. The geophysical measurements were made using a range of methods including shallow seismic refraction, vertical electrical sounding, resistivity profiling, and magnetic measurements. Figure 2.3.25 shows the measurements made along a profile following a contour on the lower quarter of the slope. The clearest evidence of the failure is provided by the SSR measurements processed by the Hagedoorn method extended by the effects of the penetration of the seismic signal below the refraction horizon.

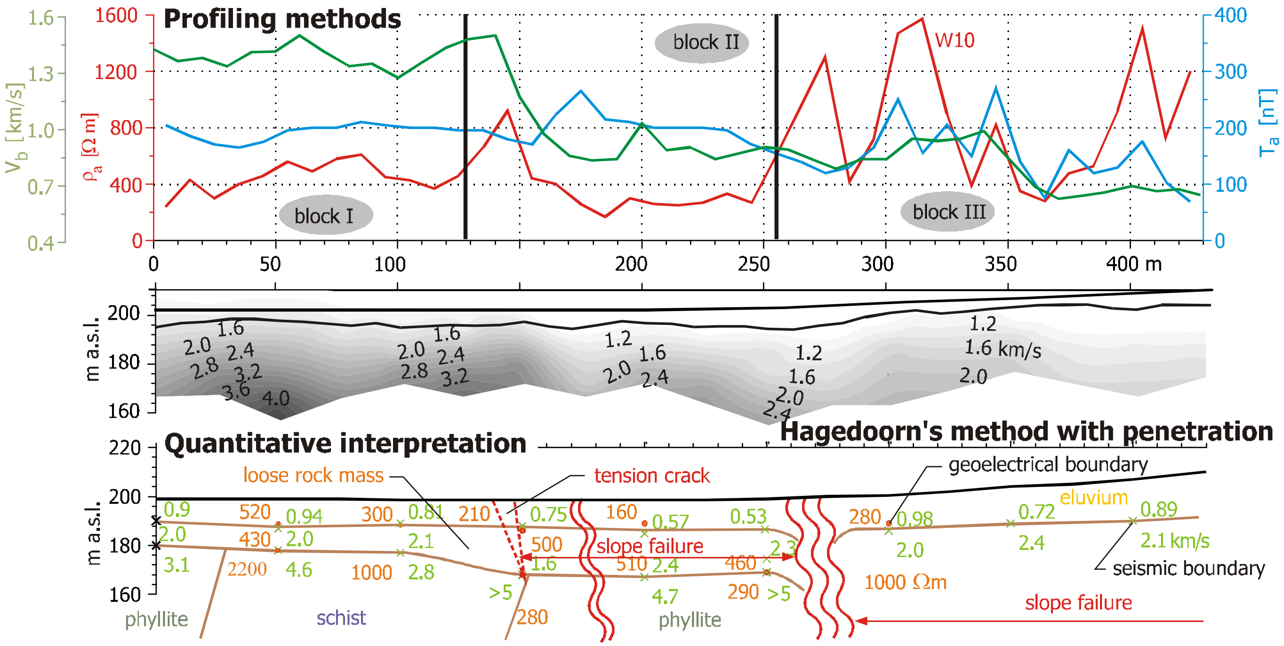


Fig. 2.3.25 Section along the Genal P3 profile showing the interpretation of the underlying geology based on the results of seismic, geoelectrical and magnetic surveys

The velocity field divides the studied rock mass into three blocks. Block I consists of intact schist, with less abundant phyllite and their weathering products consisting (from surface to bottom) of silty clay, eluvium and loose rock mass. The total thickness of these layers is about 25 metres. The most fractured mass is in block III. The velocity contours clearly show the slower growth of velocities with depth and their greater variation in the horizontal direction. In this block, based on the method of critical distances, the base of the third velocity layer must lie below 40 metres. Using this geophysical evidence, together with information from the field trips, the base of the fractured rock mass caused by the slope failure lies at a depth of over 50 metres.

The curves depicting the values Ta and ρ_a are relatively smooth in the intact mass in block I. In block II there is evidently a definite fluctuation of these curves. W10 at 145 metres shows a clear anomaly, which is the northernmost boundary of the slope failure marked by a lateral tension crack. Block III lies within the core of the zone of failure. Curves Ta and W10 here clearly show the largest variation. The course of the boundary velocity Vb is interesting. In block I the values of Vb are higher and reaching about 1.5 km/s. In block II they drop to about 0.9 km/s. The same values are also recorded in block III.

Another example is the Corojo III dam site, where a morphologically favourable profile was selected but determined to be unsuitable on the basis of the preliminary engineering-geological survey. The reasons for rejection were the existence of slope failures on both the flanks of the valley and also the extreme permeability of the karstified limestone (Fig. 2.3.26). The organogenic limestone has been karstified along their contact with impermeable rocks that also coincides with a significant zone of tectonic fracturing.

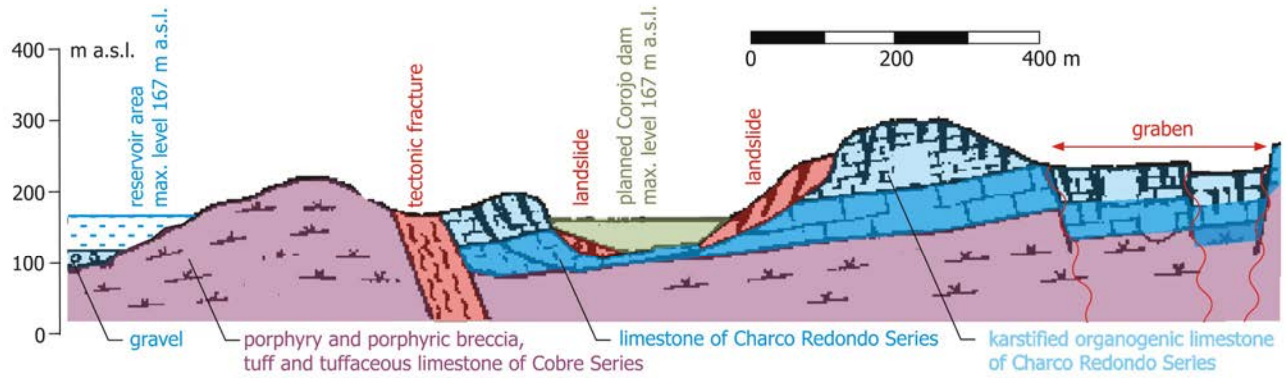


Fig. 2.3.26 Geological section along the profile chosen for the Corojo III dam, showing the underlying landslides

The Charco Redondo dam profile on the River Cautillo is yet another example. A major slope failure on the left bank of the profile originally chosen for construction of the dam meant that no dam of any type could be built for an economically acceptable cost (Fig. 2.3.27). Open cracks, 2 to 20 metres wide and 10 to 40 metres deep between the individual slipped blocks of limestone were detected. These cracks are filled with detrital limestone material, smaller blocks of the collapsed limestone mass and silty clay. The shape of the cracks, their orientation parallel to the direction of flow of the river and the nature of their fill would have led to massive losses of water from the reservoir.

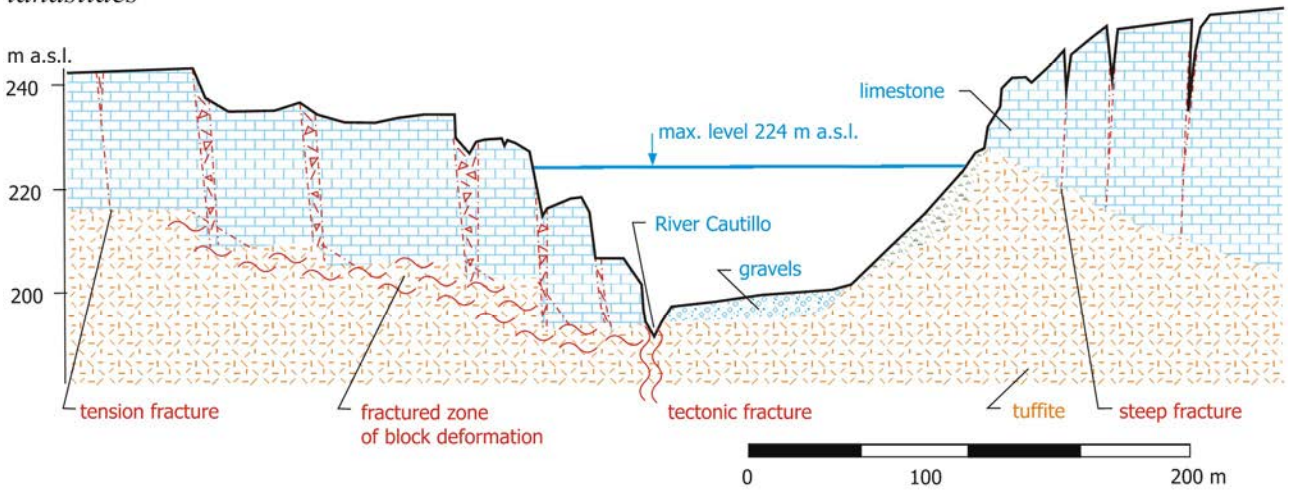


Fig. 2.3.27 Cross section showing the extensional fractures and unstable blocks of limestone on the profile of the Charco Redondo dam

As both these examples from Cuba show, it is essential during an engineering-geological survey of a dam site to be aware that an apparently favourable morphology can also have associated major problems of mechanical stability and permeability. These problems arise from the combination of climate and the geological formations that occur in Cuba. In this case, a humid tropical climate combined with acidity of the groundwater results in intensive weathering processes to considerable depths. During frequent cyclones several hundred millimetres of precipitation can fall during a few hours. This torrential rainfall causes rapid erosion of sediments and the rock mantle. In Cuba and countries in the same latitude, bioclimatic factors govern the development of topography and drainage to a large extent. Geodynamic processes include deeply incised erosion and slope failures. The underlying geology of Cuba was determined by marine transgressions during the Cenozoic, and limestone formations make up an important part of the sedimentary sequence laid down. These were subsequently affected by intense karstification. The slopes of the deeply incised valleys are affected by repeated slope failures.

At dam sites where slopes are susceptible to failure, it is an advantage to design a dam constructed from local materials or a massive concrete dam to weigh down the foot of the slopes and thereby increase their stability. The techniques used in construction must take account of the potential instabilities, and the structures must be arranged so that the slopes are not destabilized by an unsuitable excavation during the construction of the dam. For example, during the construction of a dam on the River Guisa in the Sierra Maestra

Mountains, the slope was undercut when preparing the foundation for the intake structure. As a result, a mass of Palaeogene limestone slid over Cretaceous porphyritic tuffite. In total, 800,000 m^3 of material slipped and there was an enormous delay in dam construction because all the slipped material had to be excavated and transported away from the construction site (Fig. 2.3.28).

Fig. 2.3.28 Landslide of Paleogene limestone on Cretaceous tuffites at the Guisa dam site (a photo by O. Horský 1980)

At Dalešice, the risk of a landslide, especially from the right bank during channel deepening, was also one of the important reasons why the construction of an arch dam was not recommended. For the foundation of the powerhouse downstream from the dam, excavations in the valley were designed to reach a depth of 40 metres. This would probably have disturbed the stability of both the adjacent slopes because of the unfavourable orientation of tectonic fractures in that section. Excavations were made later to test the alternative plan to build an embankment dam. These confirmed that the decision to abandon the plan for an arch dam was correct. The stability of the right bank had already been disturbed during excavations to a depth of 15 to 20 metres and it slipped into the excavation (Fig. 2.3.29). The slippage of the right bank into the foundation pit of the penstocks caused a delay in the construction of the whole structure and an increase in the costs. It was necessary to progressively remove the slipped material and to stabilize the slope failure by anchors.

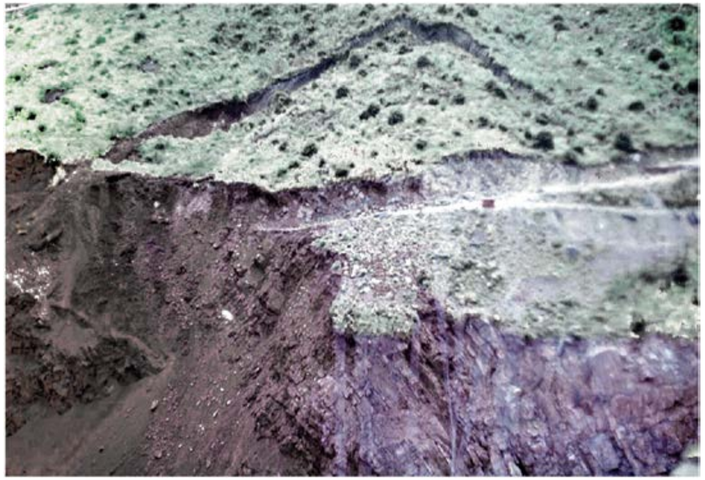


Fig. 2.3.29 View of a landslide at Dalešice (a photo by O. Horský - 1978)

The construction of lightweight buttress dams or arch dams on morphologically suitable sites in narrow valleys where there is also a danger that the slopes will fail during excavations or after the commissioning of the water-retaining structure carries serious risks. There are a number of examples where disasters have occurred, for example the explosive failure of the Malpasset dam in 1959 (Fig. 2.3.30).

The Vajont (Vaiont) dam is another example of a disaster caused by a massive landslide. In 1963, two years after the dam was completed, a landslide of over 100 Mm^3 of rock from Monte Toc collapsed into the reservoir. This caused a 250-metre high wave, which carried 30 Mm^3 of water over the dam and totally destroyed the little town of Longarone and several other villages downstream. Villages at the level of the reservoir were also heavily damaged. This disaster caused 2,117 casualties. The Vajont dam itself, at that time the highest arch dam in the world (265.5 m), was virtually undamaged by the flood wave. This testifies to the high quality of the materials used in construction and the safety of the foundation, as well as the excellent workmanship involved in the construction of the dam itself. However, the results of the survey and monitoring of slope stability in the backwater area and especially the behaviour of the slopes after the reservoir was filled were not taken seriously enough. The existence of a failure on the left slope was known and monitored, but the disaster was not predicted. A view of the dam and the landslide is given in Figure 2.3.31. A section of the rock mass at the site of the landslide is shown in Figure 2.3.32.

Figure 2.3.30 The Malpasset dam, after 50 years

In narrow valleys cut through rock, the construction of a concrete arch dam is usually considered the best solution. The morphology of the valley significantly affects the shape of the arch constructed. The narrower the valley and the deeper it is cut into the rock, the more suitable it is for an arch dam. The parameter that governs the construction of an arch dam is the coefficient α, which is defined as the ratio of the valley width at the dam crest to the dam height. The construction of an arch dam is recommended in valleys for which the ratio α is up to five subject, of course, to the geological conditions being suitable.

The arch dam built on the River Tuyère at Bromme (Fig. 2.3.33) was founded on granite in a situation where $\alpha = 2.5$. Another interesting example is the site

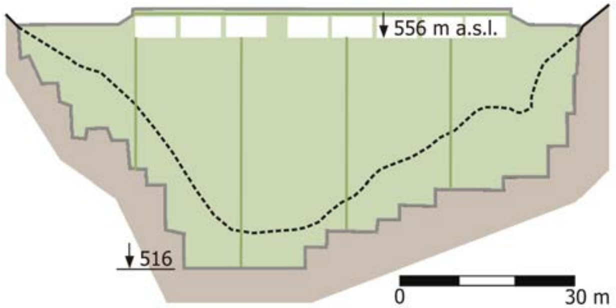


Fig. 2.3.33 Profile along the Bromme dam, (after Záruba and Mencl, 1957)

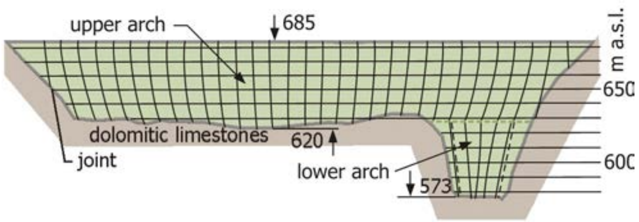
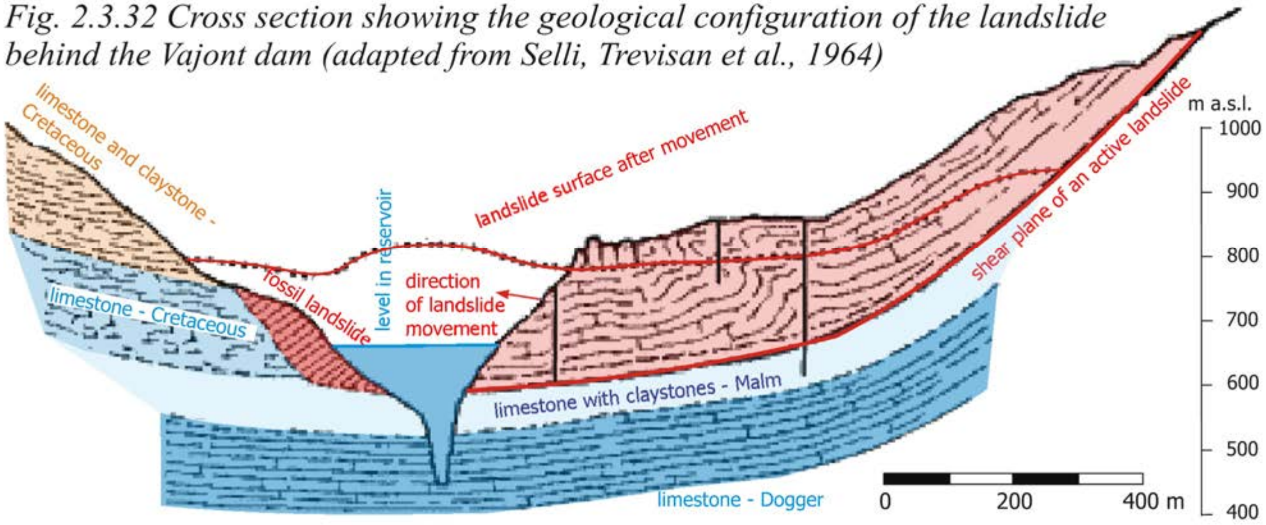


Fig. 2.3.34 Profile along the Pieve di Cadore dam (after Záruba and Mencl, 1957)

Fig. 2.3.31 View of the Vajont Dam, showing a scarp produced on Monte Toc above the reservoir as a result of the catastrophic landslide in 1963 (a photo by P. Bláha - 2008, a tale of eight photes)

Fig. 2.3.32 Cross section showing the geological configuration of the landslide behind the Vajont dam (adapted from Selli, Trevisan et al., 1964)



of the dam at the Pieve di Cadore reservoir in Triassic dolomitic limestone. The original plan was to construct a combined dam in the area of a deep erosion channel that would consist of an arch dam complemented by a gravity dam in the remaining part of the profile. However, studies showed that, for reasons of safety, it would be better to construct an arch dam divided by a contraction joint and so this was the solution adopted (Záruba and Mencl, 1957 – Fig. 2.3.34).

Figure 2.3.35 Landslide threatening dam

Slope failures can pose a risk to dams not only during their construction, but also during their operation. An example of such a risk to a dam is shown in the Jigiristan dam in Uzbekistan. It is not a classical dam designed for long-term water impoundment, but a dam the purpose of which is to protect the area below the dam against disastrous effects of slope failures. These failures in Uzbekistan are formed during snow melt, which is often accompanied by heavier precipitation. These slope failures are of the type of flow when loess becomes liquefied on the slopes with high soil moisture and suddenly collapses from the slopes to the valley where the sliding material is washed out by an increased discharge of a flood wave. Such a formed liquid moves farther in the local watercourse. The velocity of such slope movements reaches up to tens of kilometres per hour.

The Jigiristan dam 30 m high and 190 m long is constructed on Neogene sediments - claystone and sandstone (Fig. 2.3.35). The bottom of the valley is formed by Quaternary gravel and flood loam. Ancient valleys on the slopes formed by pre-Quaternary rocks (sandstone and claystone) are interwoven by a numerous network of old erosion furrows, many of which are filled by loess. During the year, the dam lake is filled by water minimally. A general view of this dam (a black-and-white photograph) shows that three slope failures slid from the left slope of the valley, one fell into the dam lake, a second slipped onto the dam itself and a third flowed to the valley below the dam where it damaged several houses. The slipping of the middle slope failure did not cause any damage to the dam body at the moment of the slope movement (4.5.1991). But it is a newly naturally formed system of water outflow from the left slope of the valley in the vicinity of the keying of the dam, which has an adverse effect. A detailed view shows that any further precipitation will cause a concentrated water outflow from the slope to those places in which the slope failure had its highest thickness before. This will lead to the intensive erosion of not only the slope itself, but also the slipped material and the material of the dam body. In general, it can be easily predicted that the dam itself could be dangerously damaged either during heavier precipitation or when the thicker snow cover melts. Therefore, it would be indisputably appropriate to accept such remedial measures which would divert the water flowing down the erosion furrow off the dam body.

Thanks to the development of computational methods that can be used to model the behaviour of proposed dam structures, the upper limit of the parameter α has been increasing in the most recently constructed arch dams so that they are now being used in much wider valleys than previously. For example, the Fedaia arch dam has $\alpha = 7$, and the Moulin Ribou dam $\alpha = 8$. In the paper by P. Peter, L. Votruba and L. Mejzlík (1967) it is stated that:

- where $\alpha \leq 3.5$ to 4.5, the conditions for constructing arch dams are very favourable;
- where α $4.5 \leq 6$, an arch dam can be designed economically, but the behaviour of the structure must be thoroughly modelled;
- where $\alpha > 6$, the arch dam design might still be considered; but
- where $\alpha > 12$, the construction of an arch cannot be justified because the volume of concrete required would be larger than in a gravity dam.

At Dalešice, where $\alpha = 3$, there was an option to build an arch dam but because of geological conditions this was rejected as unsuitable. Under conditions in which the ratio $\alpha \geq 5$, it is usually more advantageous to build a buttress dam or a concrete gravity dam. For example, the buttress dams at Svarthalsforsen, Balforsen and Krangede have $\alpha = 5$, 12.2 and 17.5, respectively. The Nant-y-Moch dam has $\alpha = 6.3$ and the Main Shira dam has $\alpha = 16.1$. For the buttress dam at Nevers, the value of $\alpha = 58.8$. It is generally considered that buttress dams can be constructed to heights of 30 to 120 metres if geological conditions permit. At a height lower than 30 metres, the costs of a buttress dam are usually higher than those of a gravity dam. If the height of a gravity dam or a buttress dam exceeds 120 metres, the volume of materials needed for construction rises substantially and upward pressures on the dam foundations also rise and this causes a reduction in the stability of the structure.

In wider valleys with slopes rising at moderate angles, the construction of an earth/rock-fill embankment dam is usually preferred if the morphology of the dam site guarantees the stability of the construction on the downstream and upstream sides and if the earthwork can be widely undertaken mainly at the dam site. Embankment dams are mostly less expensive if the main service structures can be placed directly within the body of the dam. If diversion tunnels, emergency spillways, etc., must be built separately, the construction will become much more expensive.

With regard to the stability of the dam as a whole, it is easier to ensure this in a narrow valley than in a wide open one. However, one disadvantage is the possibility of crack formation due to the differential settlement under the central part of the dam and in the areas adjacent to the slopes. The most dangerous are those cracks that cut across the dam body because water from the reservoir can readily leak through them and once a passage for flowing water is established, piping or internal erosion of material can lead to a serious failure. The sealing component must therefore be designed in such a way as to be capable of deforming plastically under stresses created by differential settlement.

There are four basic categories of profile which must be considered when constructing a dam (Peter *et al.*, 1967 – Fig. 2.3.36):

- Profiles with a V shape – suitable for arch dams;
- Profiles with a U shape – suitable for concrete gravity and buttress dams and for arch dams if the ratio of dam height to the length of the dam crest is appropriate;
- Unilaterally extended profile – suitable for gravity, buttress, rock-fill and combined dams; and
- Bilaterally extended profile above thick layers of cover formations – suitable for the construction of earth- and rock-fill embankment dams.

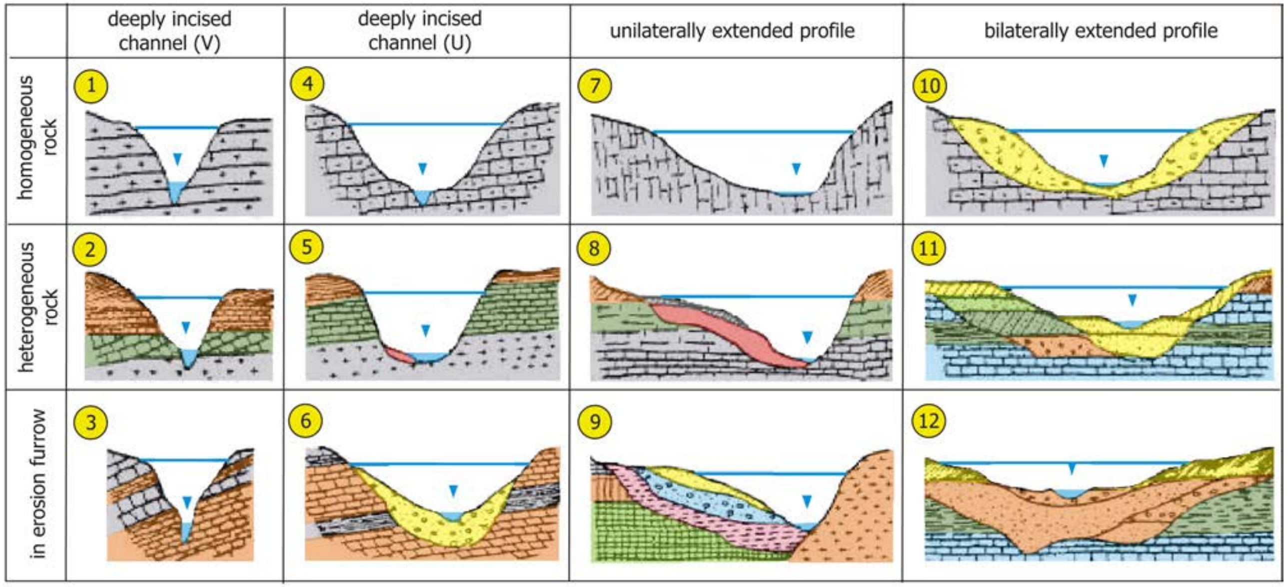


Figure 2.3.36 Common topographic and geological scenarios for dam sites (based on Peter, Votruba and Mejzlík, 1967)

The survey of the topography at the site chosen for a future dam is important not only to enable the best choice of the type and structure of the dam, but also so that the design can be related to the natural environment in which it will fit. Other important factors to consider are the construction of access roads and the suitability of sites on which construction work can be undertaken for a plant depot, the storage of construction materials, a screening plant, a batching plant, and other important installations. If detailed study of the morphological conditions at the dam site and in the backwater area is neglected, dangerous or difficult situations can result. These can add significantly to the cost of the operation and the time

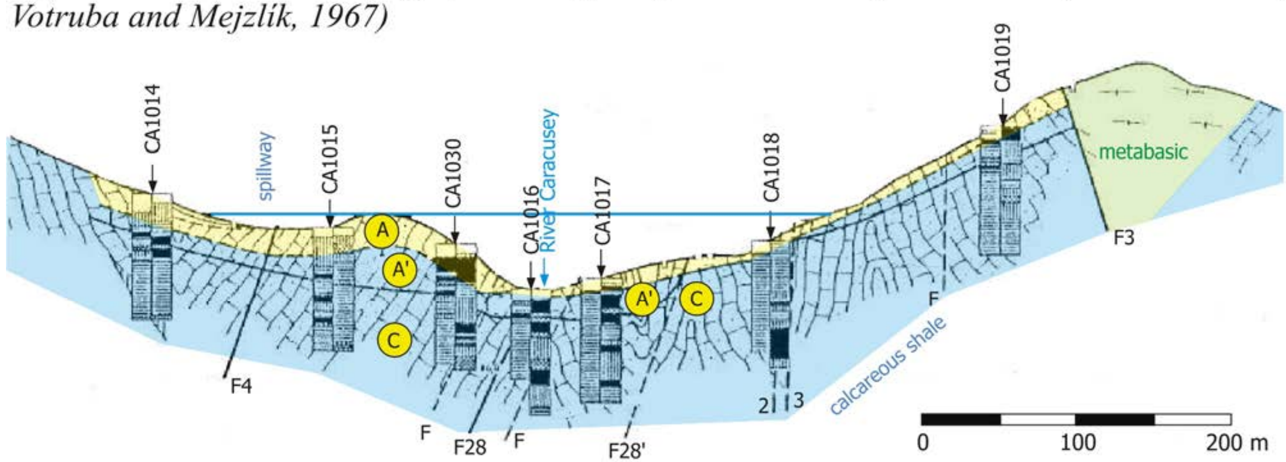


Figure 2.3.37 Geological section along the profile for the downstream reservoir of the Centro Cuba PSHEP on the River Caracusey. A) near-surface zone of stress release; A') intermediate zone of partial stress release; C) zone of normal confining stress; F) faults; 2//3) boundary between different metamorphic zones

involved, as well as placing the lives of the workforce and the public at risk. Underestimating the effects of topography and the risks of slope failure, combined with other factors, can give rise to very serious problems even after the commissioning of the dam structure.

The morphology of the site and the availability of suitable construction materials are crucial factors in determining the type of dam and the selection and layout of hydraulic facilities. For example, on the River Caracusey in the Escambray Mountains the morphology of one of the optional dam profiles enabled a spillway and a chute to be placed in a depression on the left bank where the dam was keyed (Fig. 2.3.37).

2.3.3 Effect of Climatic Conditions on the Selection of a Type of Dam

Climatic conditions at a dam site are not the decisive factor in selecting a suitable type of design for a dam but they are an important factor in determining the technology and organization of construction. Climate can have a significant effect on the time required to complete the construction of a dam. Delays in construction due to adverse climatic conditions invariably lead to increases in the cost of construction.

Sharp changes in temperature, and especially extremely low temperatures, cause changes in the stress distribution in concrete. In concrete gravity dams, cracks that form can be repaired so that they do not threaten the stability of the dam. In arch dams cracks may cause a significant redistribution of the pattern of stresses. If such changes are not allowed for in the original design, the stability of the dam can be threatened. Cracking of concrete can expose steel reinforcement that will then be subject to corrosion.

In areas with high precipitation, concrete dams are preferred because the technology of construction is simpler and floods are easier to manage. In the case of earth/rock-fill embankment dams, significant problems are caused by extreme precipitation. This has a negative impact on construction material quality, complicates the compaction of the fine material (mainly clay used for the core of the dam) and filters must often be artificially protected. This results in significant delays in the schedule of construction. Frost, snow and torrential rain can all cause interruption in the dam erection. It is either impractical or very difficult to undertake construction of a homogeneous earth-fill embankment dam of medium or great height under such climatic conditions, especially if the contract for construction places stringent limitations on the time for completion. In such cases, it is more practical to build a rock-fill dam with a relatively narrow core of clay or of artificial impermeable materials such as asphaltic concrete, plastic sheets, and the like.

2.3.4 Seismic Effects

In areas that are seismically active, all parts of a dam structure and the related facilities must be designed to take account of the forces that will act during a seismic event. This will include both natural and technical seismic effects. Each type of dam will respond differently to seismic stresses. During a seismic event, the behaviour of the dam and related structures will depend on the magnitude and frequency of oscillation produced by the event at that specific site. The response of the structure will be determined by the magnitude of the waves and on the orientation of the structure relative to the forces imposed by the event.

Arch dams are not recommended in areas affected by seismic activity because their structure is easily damaged by the stresses imposed and the movements in the rocks to which the structure is embedded. Even so, some arch dams have been built in seismically active areas, e.g., in Japan, California, Central America, Peru, and elsewhere. The principles on which buttress or gravity dams are constructed allows for some independence in the behaviour of individual components so that dams of this type can withstand minor deformations without the risk of an overall failure of the construction. In this context, movements along the direction of flow in the stream are more serious and could have a catastrophic effect.

The most suitable types of dams in seismically active areas are embankment dams. These are most resistant to the stresses imposed by seismic events. Due to the complexity of an embankment dam, the possibility of making an accurate analysis of its behaviour during a seismic event is limited. This has led to a conservative tendency in designs for dam bodies of this type. New computer techniques combined with the most recent seismological research now allow the design of a dam to be assessed in relation to the stresses imposed by seismic events. The stability of such structures has led to the highest dams in the world being of embankment type.

During the initial engineering-geological survey of a dam site it is necessary to study the geological structure in relation to technical seismicity that could be produced when the rock mass is progressively loaded by water and the dam body itself. For example, on the River Mundo, the 49-metre high Camarillas gravity dam with a backwater reservoir volume of 40 Mm^3 was founded on Cretaceous limestone without apparent complications (Fig. 2.3.38). However, after the reservoir was filled, the frequency of quakes in the area increased by four to six times and the quakes were recorded up to a distance of 100 km from the dam.

A further investigation showed that the Cretaceous limestone formed a dome underlain by Keuper marlstone and gypsum of Upper Triassic age. Due to the diapiric folding of the underlying marlstone with gypsum, the overlying limestone layers were deformed and earthquakes had been occurring even before the construction of the dam. The intensity and frequency of these quakes increased after the reservoir was filled due to the changes in the distribution of the state of stress in the surrounding rock mass. After four years, the stress situation adjusted and the frequency of earthquakes diminished.

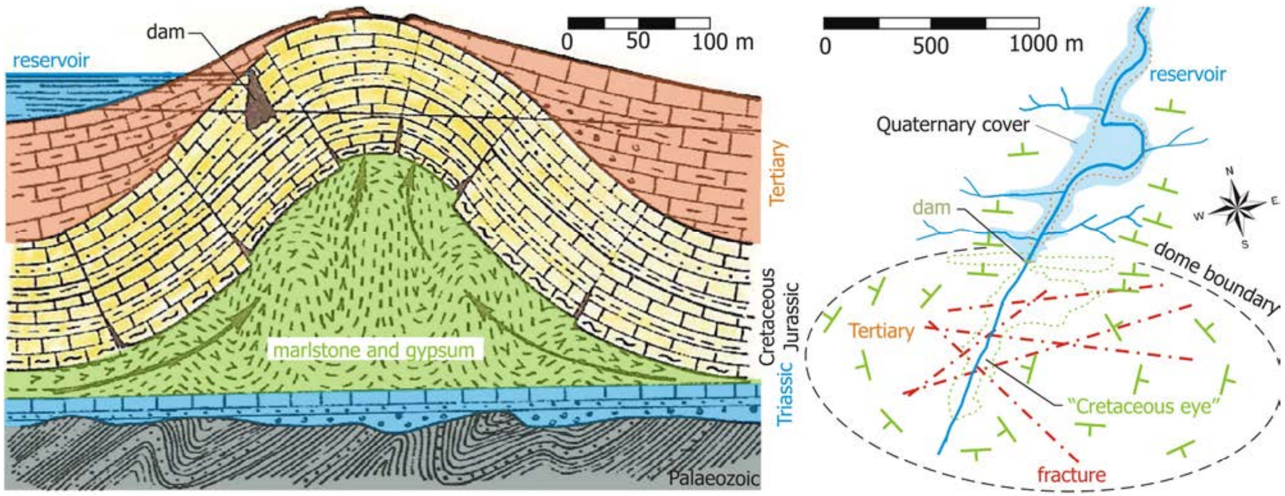


Figure 2.3.38 Geological cross section parallel to drainage showing the setting of the Camarillas dam in relation to the uplift caused by a diapir of marl and gypsum. The sketch map to the right shows the exposure of the core of Cretaceous rocks by erosion along the the river that has cut down through the rising diapir. The dip and strike of the Cretaceous strata are shown in green. Important fractures are shown in red

2.3.5 Construction Materials

Construction materials used for concrete dams must be of appropriate quality. Assessment of the suitability of material is not difficult and is defined in relevant standards. The main problem is to ensure that there is an adequate supply of suitable aggregate in the vicinity of the dam because transport over large distances would significantly increase the cost of construction.

In the case of earth- and rock-fill dams, it is necessary to ensure that the amount of material required for the construction of the dam can be obtained from local sources. Due to the fact that large volumes are required, the distance from the dam site to the source is a factor of great economic importance. In principle, it is possible to use any inert geological material for a dam fill. Such a material should not be soluble and must be dimensionally stable, i.e. its volume should remain constant. The physical parameters governing the mechanical behaviour of soils and aggregate are their strength, compressibility, workability, compactibility, permeability and resistance to weathering and frost. These determine how the material responds during construction and operation of the dam and affect the dimensions of the dam structure. If it is not possible to find materials for the construction of the sealing core in the vicinity of a planned rock-fill dam, the problem can be solved by using an asphalt concrete core (Fig. 2.3.39) or by sealing with an impermeable sheet. If, however, it is not possible to obtain sufficient aggregate for the dam structure, a serious problem does exist. The availability of a sufficient amount of material of satisfactory quality at an economically realistic distance from the dam site thus becomes one of the crucial factors in the selection of a suitable type of dam.

In the construction of dams, practical experience has shown that it is easier to overcome problems related to the foundation of the structure than to manage the situation when the properties or amounts of construction material turn out to be less than the surveyed estimates. If the cubic capacity of the reserves is not sufficient, the problems will be very serious. In cases where an adequate reserve of construction materials has not been established during the preliminary survey of a dam site, the planned schedule for construction can be slowed down so much that significant financial losses are incurred. Moreover, when a project comes under pressure to

Figure 2.3.39 View of the impermeable asphaltic concrete cover on the upstream face of the Morávka rock-fill embankment dam before the reservoir was filled (a photo by O. Horský - 1963)

Figure 2.3.40 View of the landslide that took place on the margin of a stone-quarry in the backwater area of the Dalešice dam (a photo by O. Horský - 1972)

obtain an adequate supply of rock for fill, other difficulties can arise. In the case of Dalešice PSHEP, a quarry was opened in the future reservoir area. By exceeding the criteria for the stability of the walls during excavation in this stone quarry, a vast slope failure took place (Fig. 2.3.40). This led to further problems when the slope had to be stabilized after the reservoir was filled. Of course, as a general rule, the location of the deposit of construction materials in the reservoir area is desirable because there is no need for reclamation and last, but not least, the storage volume of the reservoir will be enlarged.

2.3.6 Ecological Factors Affecting the Construction of Reservoirs

An important task in planning the construction of a reservoir in a given natural environment is to define the areas of interaction with the environment during construction and later during the operation of the dam. The relationship between a reservoir, the retaining dam structure and its wider surroundings is complex. The boundaries of interaction with the surrounding environment are not easy to define and it may take many years for re-equilibration to take place. It is easier to define the relation between the water-retaining structure and the surrounding rock mass. The changes that take place have been described by Dziewanski, *et al*., 1981, who define the interactions as shown in Figure 2.3.41.

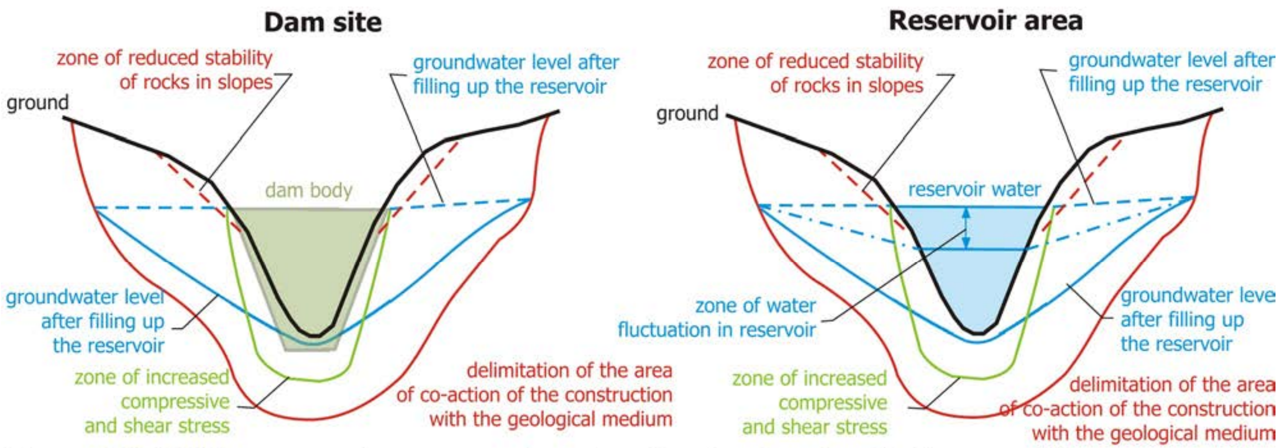


Figure 2.3.41 Diagrammatic cross sections showing the domain of influence of dam construction work on the surrounding geological environment (adapted from Dziewanski et al., 1983)

It is necessary to emphasise that when assessing the impact of the construction of a dam on the surroundings, the whole complex of the water-retaining structure cannot be considered separate from the basin itself. A dam only acts on its own in relation to its surroundings during the stages of construction. Reservoirs and dams always have a substantial impact on the natural environment and provoke criticism mainly because the social and economic benefits of the project are not fully appreciated.

In the past, the social and economic imperatives that drove dam construction meant that ecological factors and environmental impact were not properly taken into account. In particular, potential threats to certain animal and plant species, the preservation of the natural character of the landscape, or even of protected areas in the proximity of the dam structure and reservoir were overlooked. Hence, after 1989 (the Velvet Revolution), certain ecological activists were striving to empty the Mušov reservoirs at Nové Mlýny so that the rare ecosystem there could be restored to its original state. When constructing the Dalešice and Mohelno dams, detailed studies were carried out to determine the impact of such a reservoir on the serpentine steppe at Mohelno, where many rare animal and plant species are found. After commissioning the work, a definite change in the microclimate did take place, but the worst predictions concerning the destruction of the steppe environment of the surrounding landscape were not fulfilled (Fig. 2.3.42). In general, before 1989, water management and economics were the main factors that governed the strategy of construction of dams and reservoirs and minimal consideration was given to their ecological impact. Recently, the question of ecological impact is treated with the same priority as economic, social and engineering factors. In some cases, ecological concerns are given precedence over other aspects of a project so that it becomes very difficult to reach a consensus on the decision to build a dam. For example, in the 1970s, an engineering-geological survey was carried out for a dam project at Nové Heřminovy (Jeseníky Mountains). Nowadays, approval for this project would be difficult to obtain, even though it would significantly reduce the risk of serious flooding.

Figure 2.3.42 Schematic plan depicting the position of the Mohelno and Dalešice dams in relation to the surrounding topography, settlements, roads and the Mohelno Serpentinite Steppe

Ecological factors play a very important role in the location of reservoirs, particularly at the present time. However, even though the site of the dam and its height may be determined by ecological considerations, the design and type of construction used and the ancillary works are determined primarily by civil engineering factors.

In some countries, huge projects have been designed and carried out without taking account of the extensive and irreversible changes that can occur. As a result, the ecological impacts can be disastrous. Warnings were given by experts about the effects of constructing the Three Gorges dam on the River Yangtze. This is already partly in operation and the severity of the impacts is already apparent. A list of hazards has been published, including catastrophic erosion and landslides on the steep slopes surrounding the reservoir, silting of the reservoir, extreme growth of aquatic algae, induced seismicity, the degradation of the aquatic ecosystem, and conflicts because of the lack of land for millions of displaced inhabitants (see Chapter 9.5).

Another topical problem relates to the closure of an operating dam. It is necessary to ensure that the section of a valley affected by the construction of dam and a reservoir can be restored to a trouble-free condition after the dam ceases to operate. Before closure, it is necessary to carry out a study to determine how to proceed in order to prevent subsequent ecological damage. In such cases, it is necessary to assess the impact of the decommissioning of the reservoir on the environment, and to plan management of the landscape and protection from flooding, as well as ensuring the quality of ground and surface waters.

2.3.7 Environmental Impact Assessment (EIA) in Relation to Construction and Operation of Water-Retaining Structures (Adapted after Čáslavský M., 2009)

In countries with established environmental legislation, or with an awareness of the principles of sustainable development, it is mandatory to assess any dam construction project in relation to its impact on human health and the environment. This assessment is an important procedure for preventing subsequent damage. In countries where this procedure is not imposed by legislation, it is appropriate to undertake such an assessment voluntarily, at least in a basic form, so that the immediate effects on the surrounding environment are taken into account. Serious environmental impacts can thus be foreseen in advance. The cost of repairing environmental damage could significantly exceed the economic benefits of a water reservoir project if financial resources are limited.

In preparing EIA documents, the effects on public health and the environment must be assessed comprehensively taking into account animals and plants, ecosystems, soil, rocks, water, air, climate and landscape, natural resources, tangible assets and cultural monuments. The principal purpose of the environmental impact assessment is to provide the information required to make professional decisions in relation to regulations governing planning and construction permit procedures and to facilitate dialogue between the contractor, local government and the local inhabitants about issues such as land ownership, agriculture, traditional hunting and fishing rights, disturbance of patterns of animal migration or grazing, etc.

The EIA must be an open and fully coordinated procedure involving professional institutions and the public. Attention must be given to those components of the local environment which will be most affected by the construction of a reservoir and the retaining dam, taking into account the impacts on the life of the local inhabitants and the net benefits to be obtained by covering a part of a terrestrial ecosystem with water. The assessment will incorporate the basic technical data relating to the dam project, as well as taking account of links with other projects and the social and economic rationale that led to the choice of its location. The EIA will also include the anticipated date for the start of construction and its completion, a list of the self-governing territorial units that will be affected and the consecutive decisions that must be taken.

Information on inputs and outputs connected with the implementation of a project is essential. The important inputs are the type of soil and the category of protection, the area of land to be appropriated, sources of water and anticipated consumption, the amounts and types of resources of other raw materials and energy required for the project and their anticipated consumption, and demands for transport that require construction of roads and other infrastructure. The outputs include an assessment of the effects that the project will have on the air. This will list the sources of pollution, the type and quantity of harmful substances emitted, and the methods used to suppress them and their efficiency. Wastewater will also be assessed in terms of the quantities produced, the points of discharge, and plants for treating the resulting pollution, and their efficiency. An overview of the sources of wastes, the types and quantities of waste produced, and the methods employed to handle waste will also be included. Other outputs that have a significant impact must also be assessed. These include noise, vibrations, radiation and odour. The evaluation should also contain supplementary information about changes in the topography produced by preparatory works and landscaping.

The EIA proceeds from the existing state of the environment in an affected area and includes:

a) A list of the most important environmental features in the affected area. These will include sensitive or rare ecosystems, specially protected areas, nature parks, popular landmarks, areas of historical, cultural or archaeological significance, densely populated areas, areas of contemporary ecological stress or those with old ecological loads (e.g., mine waste) or areas of unusual use.
b) Characteristics of the current state of the environment in the affected area. Here, attention is focused on the air and climatic conditions, surface water and groundwater, soil, bedrock and natural resources, fauna, flora and ecosystems, landscape, population, tangible assets and cultural monuments.
c) An overall evaluation of the quality of the environment in the affected area in relation to its sustainable load. And
d) Comprehensive evaluation of a project for the construction of a dam and its impact on public health and the environment should therefore contain a description:
   - The predicted effects of a project on the population and the environment and an evaluation of their scale and significance, taking account of specific socioeconomic impacts;
   - The predicted effects of a project on the air and climate, on the noise situation and, if applicable, other physical and biological impacts, on surface water and groundwater, soil, bedrock and natural resources, on fauna, flora and ecosystems, landscape, tangible assets and cultural monuments;
   - Possibilities of trans-boundary effects in both directions;
   - Environmental risks arising because of possible emergencies and non-standard states;
   - The measures for prevention, elimination, reduction, or compensation of adverse effects on the environment;
   - The methods used in forecasting the impacts of the project and the initial assumptions on which the impact evaluation is based; and
   - The limitations in knowledge and any uncertainties that affect the objectivity of the assessment.

If several different options for the design of a dam construction project are proposed, it is necessary to carry out a separate evaluation for each of the proposed options. The completed EIA for a project should be assessed subsequently by an independent panel of experts who have no material or financial interest in the project. This is to ensure that any recommendations affecting the decision to proceed with the project are completely objective.

Environmental impact assessment in the European region is based on the following EU regulations:

- Council Directive of 27 June 1985 on the assessment of the effects of certain public and private projects on the environment (85/337/EEC);
- Council Directive 97/11/EC of 3 March 1997, which amends Directive 85/337/EEC on the assessment of the effects of certain public and private projects on the environment; and
- Directive 2001/42/EC of the European Parliament and of the Council of 27 June 2001 on the assessment of the effects of certain plans and programmes on the environment.

In countries where there is no existing legal framework for the preparation of an assessment of the effect of a proposed water-retaining on the environment, the European directives can be used as an instruction for preparing such an assessment.

2.4 The Itaipu Dam as an Example of How to Solve Problems of Large Dams

The Itaipu hydro-engineering project (Fig. 2.4.1) will serve as an example to show a wide range of problems that are encountered during construction of such structures. It is natural that with increasing sizes of constructions it is necessary to deal with increasing problems, both in terms of number and size. The fact that this structure was built properly is also demonstrated by the circumstance that from its commissioning in 1984 to the end of 2013 it was necessary to solve only a single serious problem. This, however, was not related to the hydro-engineering structure itself, but a defect occurred in the Rio de Janeiro distribution network. At the initial stage of dam construction in 1971, two sites of construction were under consideration. One was the surroundings of the isle of Itaipu and the other at the little town of Santa Maria farther downstream. In the end, the site of Itaipu was selected, particularly because of a better possibility of rerouting the River Paraná during construction. The hydro-engineering structure lies on the border between Brazil and Paraguay on the River Paraná, at a distance of 13 km upstream from the international bridge “Puente Internacional” connecting the towns of Ciudad del Este and Foz do Iguaçu. The name “Itaipu” means in English “the sounding stone”.

Fig. 2.4.1 The Itaipu dam

The entire hydro-engineering structure is unique in a whole range of specifics. The mightiness of the hydroelectric dam called for certain unusual designs. These can be summed up in the following points:

- The construction of the dam away from the narrowest point of the river stream in the area of the planned project site;

- The complicated pathway of the dam;
- The use of several types of dams;
- The rerouting of the river during construction;
- The construction of a small dam on a right-bank tributary for supplying the main construction with electric power;
- The tremendous velocity of dam reservoir filling; and
- The shape of the reservoir bottom.

The hydroelectric power plant Itaipu is the second largest hydropower plant in the world in terms of the installed output 14,300 MW. In 2012, this was a hydropower plant with the largest annual amount of produced electric power in the world (98.3 TWh). Twenty Francis turbines are mounted in the power plant, each with an output of 715 MW. According to a bilateral intergovernmental agreement between Brazil and Argentina, eighteen turbines at maximum operate simultaneously and two always serve as a reserve.

The dam lake is 170 km long and occupies an area of 1,350 km^2. Of the given total size of the water area, 580 km^2 belong to Paraguay and 770 km^2 to Brazil. The volume of the impounded backwater with its rise to a height of 220 m a.s.l. is given by a value of 29 km^3, of which 19 km^3 is its usable volume. Because of the large flow rate of water during the rainy season, the reservoir was filled in mere fourteen days. The construction of the hydropower plant was commenced in 1975 after a treaty on the status of the structure of a supranational independent company, belonging to neither of the states, had been signed between the two countries. The construction was completed in 1982.

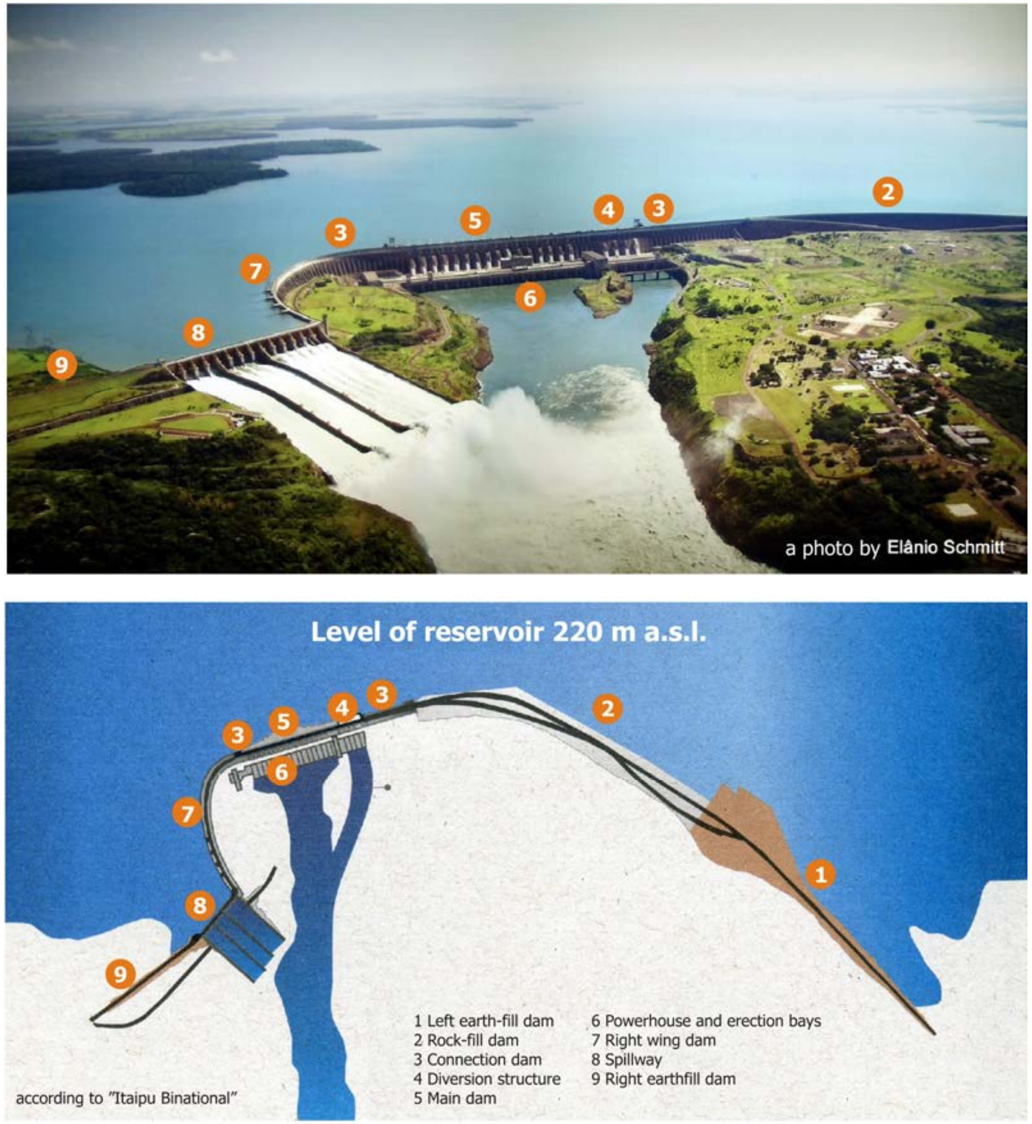


Fig. 2.4.2 Construction of the Itaipu dam

When the dam reservoir was filled, the famous Guairá Falls were submerged. The construction of the reservoir called for the appropriation of 700 km^2 of virgin forest and about ten thousand families were resettled from the backwater area. These losses are the toll for this so much-needed source of electric power, which in Brazil covers 17.3 % and in Paraguay even 75.2 % of their national consumption. Within the reservoir filling, an operation called “Mymba Kuera” took place for the rescue of the local ecosystem. More than seventeen million seedlings of different species of flora have been planted around the Itaipu dam lake and about one hundred species of animals have been relocated. Now more than one hundred and seventy species of fish live in the water of the reservoir.

Approximately 40,000 employees worked on the construction and the project cost 11 billion dollars (in then prices). Brazil and Paraguay share the generated power equally. Because Paraguay does not consume its energy in full, it transfers it to Brazil, hence partially paying off its debt. The maximum height of the reservoir dam is 196 m and its length is 7.92 km. The complicated topography of the terrain, as well as economic reasons and the possibility of shortening the time of construction led the designers to an unusual design of the dam construction. The entire object of the dam body consists of nine subunits; it is possible to describe them from east to west (Fig. 2.4.2): left earth-fill dam (Fig. 2.4.2 - 1), rock-fill dam with clay core (Fig. 2.4.2 - 2), connection dam (Fig. 2.4.2 - 3), diversion structure – dam of the River Paraná rerouting (Fig. 2.4.2 - 4), main concrete lightweight gravity dam (Fig. 2.4.2 - 5), power house (Fig. 2.4.2 - 6), right wing dam – buttress dam (Fig. 2.4.2 - 7), emergency spillway (Fig. 2.4.2 - 8) and right earth-fill dam (Fig. 2.4.2 - 9).

The geological structure of the dam site and the backwater area is characterized by a vast complex of sub-horizontally lying basalt rocks. It consists of extensive sheets of basalt lava (plateau basalt) associated with linear eruptions. They represent the largest known effusions on the Earth's surface. Plateau basalt in the basin of the River Paraná is among the largest in the world with its area 8 million km^2. Compact basalt is interlaid there by vesicular and amygdaloidal basalt and breccia. These locally form lithological discontinuities and places of

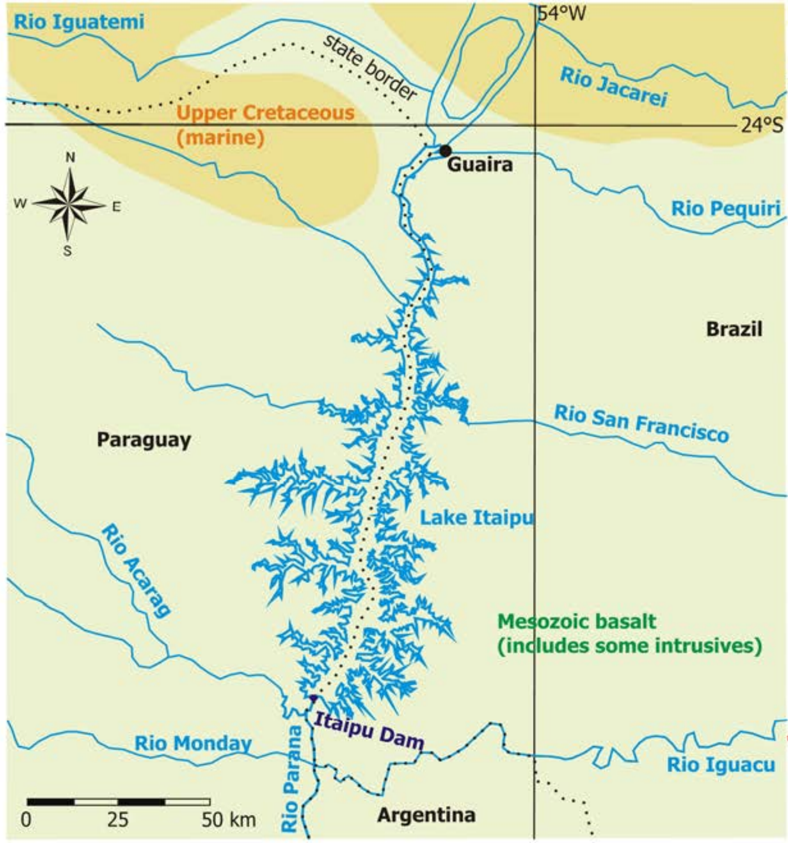


Fig. 2.4.3 Geological map of Itaipu lake surroundings (according to Geological Map of South America 1950, Geological Society of America)

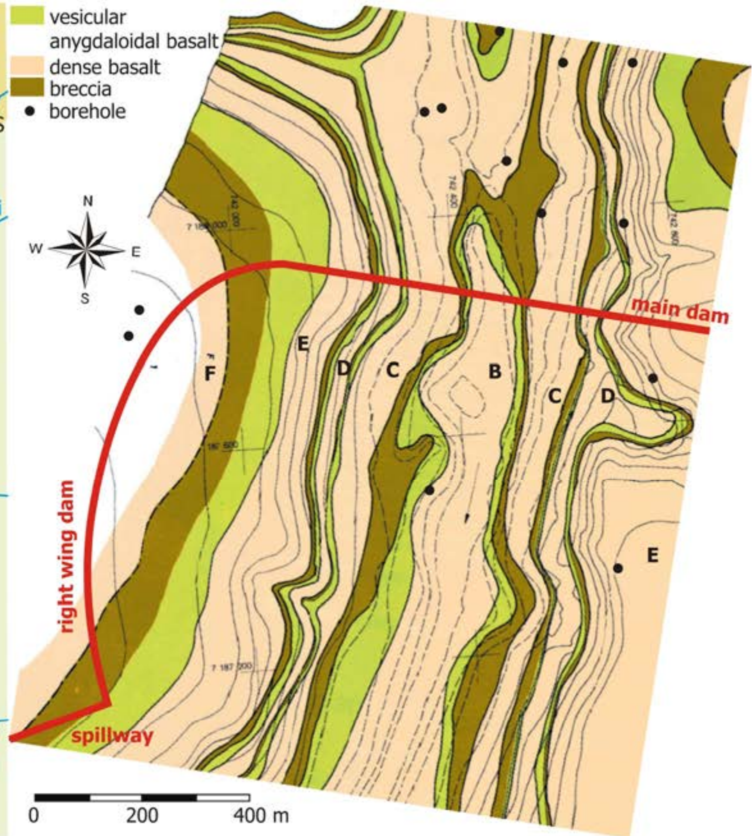


Fig. 2.4.4 Detailed geological map of Itaipu dam (according to Itaipu Binational)

weakness. The thick basalt sheet belongs to the geological formation Serra Geral, also designated sometimes as “Trapp do Paraná” of Lower Jurassic age. The basalt was formed as effusions from tectonic faults of the Mid-Ocean Ridge and its related perpendicular faults, which reached as far as the coast of the then continent (Fig. 2.4.3). The areas of the individual effusions reached even over a million square kilometres and a thickness of 30 to 70 metres. Based on the latest research, the total thickness of the basalt sheets is assumed to be 1,500 metres.

Inhomogeneous intercalations of breccia between basalt sheets are thick from 1 to 30 m; they are heterogeneous, remoulded and deformed more than basalt. Discontinuities parallel to the basic structure are mostly formed by transition zones between lava flows of different age (2.4.4). Due to tectonic unrest, the sheets were lifted above the water surface. Between subsequent effusions during the periods of volcanic quiescence, the lava solidified irregularly on the one hand, and the surface layer was exposed to weathering and erosion by the effect of water, wind and other exogene processes on the other. Sediment load and mudflows then settled on the surface of the rock mantle and thus a heterogeneous layer gradually formed before another volcanic effusion, which then partially incorporated sandy beds and clayey layers in its lower part. The horizontal permeability of the layers is many times higher than the vertical permeability. Five basalt sheets occurring in the substrate of the dam are designated by the letters A, B, C, D and E. (Figs. 2.4.4 and 2.4.5). Thanks to the deep erosion of the River Paraná, the subhorizontal areas of discontinuity were also recorded well.

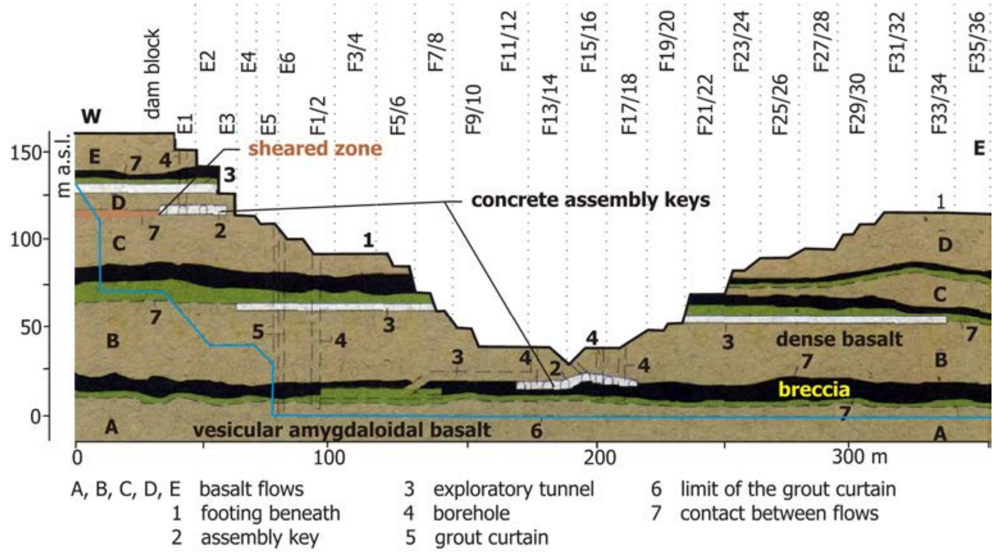


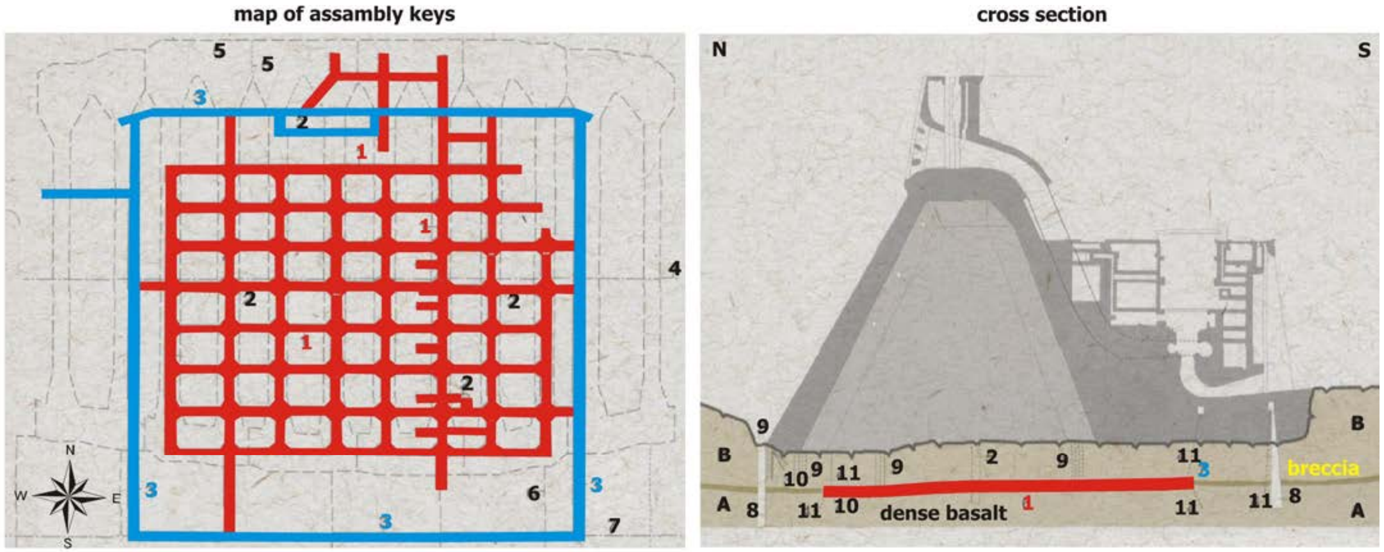
Fig. 2.4.5 Cross section of Itaipu main dam (according to Itaipu Binational)

In relation to the extent of the project and the mightiness of the dam, it was necessary to carry out perfect engineering-geological and geotechnical surveys. The engineering-geological survey began with the study of aerial photographs, which passed into engineering-geological mapping. The basic investigation was followed by drilling in conjunction with *in-situ* tests. Work continued with mining, which enabled other tests to be carried out in the field. Naturally, extensive laboratory tests were made in the single- and three-axis regime. Thus, the detailed geological structure of the area affected by dam construction was identified and the required physical, geomechanical and hydrogeological parameters were measured. These were first needed for geomechanical modelling and subsequently for dam design.

The basic geotechnical characteristics of massive basalt in the basement for the most part fully complied with the construction of the reservoir dam; also the permeability was within acceptable limits. Basalt breccia occurred only exceptionally, which is characteristic in the dam substrate by mechanical crushing rather than by subsequent hydrothermal alterations and weathering. A view of the breccia in one of the inspection tunnels is shown in Figure 2.4.6. Fortification grouting was made at the sites of important structures. The depth of the grout curtain reached down to 120 metres (Fig. 2.4.5). During construction, excavated material from the foundations of concrete structures was fully used for filling lateral dams, by which economic and time savings were achieved. The earth-fill dams on the right and left banks are constructed from clayey soils originating from excavations for the foundations of the rock-fill dam and from nearby sites for the excavation of local building materials.

Fig. 2.4.6 Breccia at the footing beneath (a photo by P. Bláha 2013)

At two sites where unsuitable mechanical parameters of the rock mass were detected, measures were taken, which improved the properties of the rock mass, particularly resistance to shear stress. These measures pertained to two sites where clays were locally situated, namely beneath the blocks E1–E3 of the lateral dam, and beneath the blocks F12–F18 of the main dam (see Fig.2.4.5) because of shear deformation at an elevation of 20 m a.s.l. At the beginning, more options of how to improve the properties of the rock mass were taken into consideration, e.g. deepening of the excavation, digging of a cutting, washout of unsuitable materials and their replacement by grouting and, if possible, use of a network of fortifying tunnels. Finally, excavation of tunnels perpendicular to each other in the dam substrate was selected for increasing shear parameters. These were subsequently filled with reinforced concrete, which guarantees that the foundation rocks are sufficiently secured against the possibility of shearing caused by the weight of the structure (Fig. 2.4.7). This option enabled these tunnels and the blocks of the dam to be constructed simultaneously; it was by 20 % cheaper than other options. 3 x 3 tunnels were excavated beneath the blocks E1–E3 and 8 x 8 tunnels beneath the blocks E1–E3. The tunnels were excavated parallel and perpendicular to the valley of the River Paraná. The cross-section of the tunnels was 3.5 x 2.5 metres, their total length was 2,600 metres and they were filled with



A, B basalt flows
1 assembly key
2 exploratory pit
3 perimeter drainage tunnel
4 axis of the dam
5 block gravity dam
6 block casa de forsa
7 axis casa de forsa
8 grout curtain
9 contact grouting
10 fortification grouting
11 drainage borehole

Fig. 2.4.7 Site repair of bad mechanical properties reinforced cement. Some fractures and rocks with poorer properties were replaced by concrete. The contact between the reinforced concrete and the rock mass was fortified by grouting. A drainage tunnel was excavated around the network of tunnels, which isolates the repaired part and reduces the piezometric pressure in this area.

The unusual geological structure of the periphery was also indicated in the backwater area. Over the entire time of the operation of the Itaipu hydro-engineering structure, no significant changes in the banks of the reservoir have been recorded. The position of basalt and its good mechanical properties are the guarantee of the stability of the reservoir banks also in subsequent years. The dam lake, however, has one peculiarity. In most dam reservoirs, their bottoms evenly rise from the dam to the end of the reservoir. But this is not the case of the Itaipu reservoir. First, the bottom of the dam lake slowly rises against the original flow of the River Paraná up to the kilometre 170 (Fig. 2.4.8). From this kilometre up to the town of Guaíra, the reservoir bottom sharply rises. In this section, the submerged waterfalls of the same name are situated, having had the highest amount of flowing water. The total height of the waterfalls consisting of 18 cataracts was 114 metres, with the highest 40-metre-high waterfall. On 27 October 1982 the Itaipu reservoir was fully formed and the waterfalls disappeared underwater. Later the submerged rock walls of the waterfalls were destroyed by dynamite to support a safer navigation on the river. The rough shape of the reservoir bottom in the figure is due to the fact that the shape of the bottom is plotted on the connecting line Itaipu – Guaíra and not along the flow of the River Paraná.

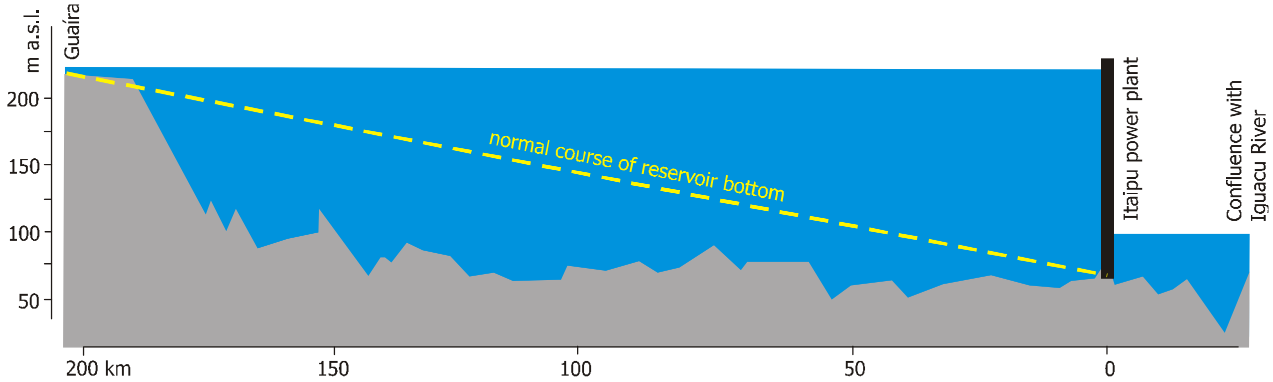


Fig. 2.4.8 Shape of the bottom of the Itaipu lake

All structures of the hydro-engineering scheme are carefully monitored. An extensive set of check measurements is used to monitor the dam body and its substrate. 2,139 sensors are mounted in the dam and its surroundings, monitoring the mechanical characteristics of the floors and walls, temperatures of the dam and its surface, pressures, deformations of the dam, the regime of groundwater and its pressure, seepage, seismicity, and geodetic measurement. More than 170,000 data are obtained from different sensors in one year. The whole monitoring is understood as a set of procedures which monitor the dam and its behaviour, control safety conditions and prove the validity of hypotheses and computer methods used in the project. If necessary, remedial measures are taken.

Based on the current state of assessment of the impact of such a hydro-engineering structure on the environment, it is very likely that the construction of such a dam would cause a massive movement against its execution. Perhaps the deepest reservations would be caused by the inundation of the Guaíra Falls, and other reservations would be towards the resettlement of 10,000 families. A certain solution could be a hydro-engineering structure built in the style of the Niagara Falls, i.e. the diversion of a part of water from the main stream through turbines. This considered solution, however, would not bring such an output of a power plant as the current hydro-engineering structure has.

3 Methods Used to Carry Out an Engineering-Geological Survey

In order to provide the information on which the design of a dam will be based, it is necessary to carry out an engineering-geological survey, the details and scale of which depend on the level of information required by the planned design, the type, size and purpose of the dam and its related structures and last, but not least, on local geological conditions. The basic survey methods include engineering-geological mapping, a geophysical survey, drilling, tunnelling and test excavation work. In addition, a hydrogeological survey must be carried out, together with studies of the physical and mechanical properties of the soils and rocks, including determination of their permeability, using standard field and laboratory tests. The engineering-geological survey is a progressive and continuous task, beginning with the analysis of existing documentation, followed by field and laboratory investigations and culminating in the stages of construction of the dam itself. Ultimately, after commissioning, the performance of the dam itself must also be monitored.

3.1 Tasks of the Engineering-Geological Survey

An engineering-geological survey for a hydraulic structure provides the engineering-geological data and information required to make decisions about the most suitable type of dam for the chosen site, its design and the ancillary facilities necessary for the operation of the project. The survey must also provide the information that will enable rational and safe working procedures during the construction of the dam and the prediction of potential instabilities caused by the load imposed by the dam structure and the water in the reservoir.

Based on these requirements, the engineering-geological survey should provide the following information:

a) The results of the engineering-geological survey must be adequate to allow the objective evaluation of engineering-geological conditions that will be faced during construction work. The designer must be provided with sufficient information to enable objective evaluation of the various options available for siting necessary constructions and plant, including the time required for their construction within the planned schedule of the project.
b) The engineering-geological survey must provide basic data relating to the physical and mechanical properties of the rock mass and its permeability, so that the project can be safely designed in relation to the prevailing natural conditions. At the same time, the results of the survey must provide the data on which safety measures to be taken during construction and after commissioning of the dam structure will be based.
c) The survey must provide the information required to predict and evaluate changes in engineering-geological conditions that will occur during construction and when the dam and reservoir are in operation so that geodynamic instabilities in the area of the construction site or on the banks of the reservoir can be predicted.
d) The survey must identify adequate resources of construction material suitable for construction of the dam and associated facilities. The excavation of earth and rock must also be planned rationally in relation to the construction of the dam.

e) The survey must warn of adverse effects that may hinder work during construction or that might cause deterioration of the environment after the dam is commissioned.

The details of the engineering-geological survey are determined by the stringent requirements set by the design of the dam itself. The construction must be founded on structurally stable rocks which have relatively low compressibility and low permeability. It is therefore necessary, if possible, to avoid unfavourable geodynamic phenomena such as slope failures, karst, rock falls, areas of high seismic activity, etc.

The methods used and the scope of an engineering-geological survey for a dam structure are determined by a complex range of factors. The local engineering-geological and topographic conditions must be assessed in relation to the type of construction, its dimensions, and the purpose of its design. Sometimes, the scope of the survey and the methods employed will also be governed by limited funds or a demanding deadline for construction. In such cases however, it is necessary to be very careful, bearing in mind that time saved in the engineering-geological survey may ultimately lead to an increase in the cost of the work itself, or to a serious accident.

Construction and operation of a dam and the associated reservoir will change the conditions in the surrounding area. Therefore, it is essential to make an engineering-geological and hydrogeological assessment of the impact of the construction and the work itself on the stability of the surrounding environment after its completion.

3.2 Rules Governing the Procedures Used in an Engineering-Geological Survey

The success of an engineering-geological survey for a dam site depends on the correct organization, performance and evaluation of the consecutive stages of the survey. The objective of these stages is to reach an optimal and rational design solution for the dam in relation to the climate, topography, and geology of the area concerned. Choice of the most appropriate site for the dam, the most practical type of construction to be used and the maximum height of the water in the reservoir are fundamental decisions that must be taken during the first stages of the survey. Subsequently, decisions on the details of design, the components of the structure and the resources required at different stages of construction will be necessary and the data required for this must be gathered systematically. This step-by-step procedure ensures the technical and economic efficiency of the survey in relation to the project as a whole.

3.2.1 Rule of Step-by-Step Procedure in Carrying Out an Engineering-Geological Survey

The successful solution of complex problems arising during the design and construction of a dam and the ancillary works is only possible if there is close cooperation and continuous communication between the designer and the engineering geologist. During the design process, there will be continuous technical input from the engineering-geological survey. At critical stages in this process, the designer presents questions and tasks to the engineering geologist, and the answers provided by the geologist are fed back into the design. Progressive advances in the level of design of the dam depend on the successful completion of each stage of the survey. At each level, specific questions are asked and progress to the next level can only take place if the necessary answers are provided.

When carrying out a survey in stages, the scope of work and the methods used are defined by the goals of that stage, bearing in mind that the work must be carried out in such a way that it can be used as part of the platform for subsequent stages of the survey. The scope of the work undertaken at a specific stage does not, as a rule, go beyond the requirements of that stage. The results obtained and the conclusions drawn at earlier stages will, when appropriate, be used to support the work carried out in later stages and the conclusions presented at each stage must contain proposals for the work to be carried out in the next stage, if there is one.

According to the requirements of a given project, or at the request of a designer, client or investor, the stages of a survey can be divided into sub-stages. Conversely, some stages of the survey can be amalgamated if the essential information relating to engineering-geological and hydrogeological conditions can be obtained from archive sources and verified by field reconnaissance. There may be other good reasons why a particular stage of a survey need not be carried out in full. Such changes in procedure, however, must not be allowed to lower the technical standard of the survey and the conclusions should have the same validity as those drawn on the basis of a complete step-by-step procedure.

Long experience shows that the best results are achieved in cases when all stages of a survey are carried out by a single organization or, as the case may be, under the direction of a single manager. When proceeding to a more advanced stage of the survey, time lost in reviewing the results of all previous stages is reduced to a minimum. If the survey is managed by a single team familiar with the whole site and its geological structure, the project will proceed efficiently from one stage to the next and both time and money will be saved. It must be understood that it is not possible to record all the technical subtleties of a given project in written reports. The reports filed will contain a clear description of specific observations that lead to conclusions and recommendations regarding the more advanced stages of the survey.

3.2.2 The Rule of Comprehensiveness of an Engineering-Geological Survey

It is a rule that an engineering-geological survey must be comprehensive. It must be ensured that all the work necessary to provide the technical information required for a given project has been carried out to best relevant standards using the most up-to-date scientific knowledge and technology. In order for the survey to answer the questions relating to the project as fully as possible, there must be close cooperation between the different specialists involved. The engineering geologist must work in close liaison with geophysicists, geomorphologists, hydrogeologists, and specialists in remote sensing and geographic information systems. There must also be close coordination between the geotechnician, the petrographer, the seismologist, the geochemist, and the specialists in hydraulics and hydrology, and with others when necessary. At present, the use of digital databases, sophisticated methods of mathematical modelling and monitoring over the whole duration of a survey is quite usual. The results obtained should also be applicable to other branches of scientific or economic activity, for example, in regional geological or hydrogeological surveys and economic geology. The rule of comprehensiveness should be applied at all stages of the engineering-geological survey.

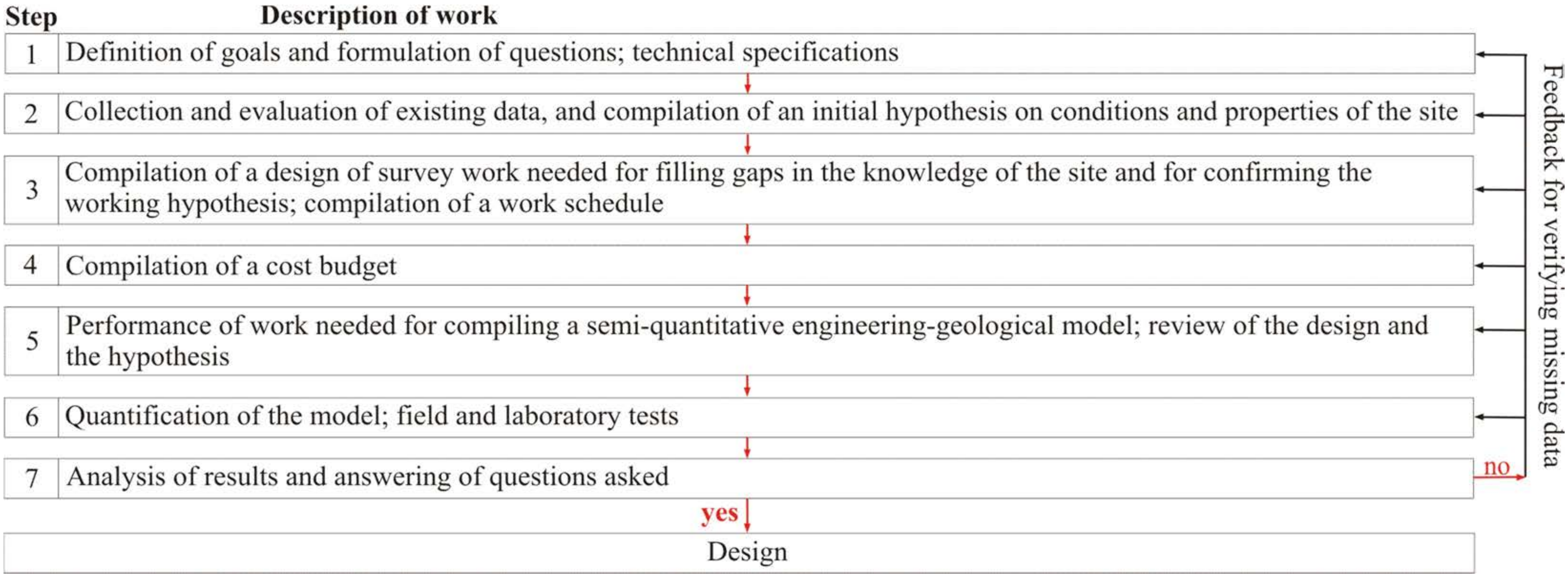
3.2.3 Accuracy, Detail and Economy of an Engineering-Geological Survey

The construction of dams and the hydrotechnical infrastructure associated with them depends primarily on a detailed and accurate knowledge of the engineering-geological conditions at the selected site. Without this information, the demanding criteria for design and construction could not be met and the cost of the project could not be properly constrained. A complete survey depends on accurate data obtained from pre-existing sources, from direct exploratory workings, and from field and laboratory tests. In order for the optimum combination of survey methods to be applied at the appropriate stages of a survey, the goals of the project must be fully defined by the client and investor in discussion with the engineer-designer and the technical staff responsible. The detail of a survey will depend on the variety of methods employed and the frequency with which measurements must be made. In engineering-geological mapping, the detail depends on the scale and accuracy of the topographic base maps and plans available. This, in turn, will depend on the quality of the existing topographic surveys, though rectified satellite images and digital topographic models can often be compiled in advance of a project. The actual engineering-geological survey of the construction site will ultimately require a decision regarding the number and type of test excavations, the holes to be drilled and the pattern in which they will be spaced.

The step-by-step procedure is crucial in order to carry out a comprehensive survey, to make effective use of the funds available, and to ensure the efficient use of the information acquired. In order for the survey to be effective, all work carried out before the commencement of the project must be reviewed so that existing knowledge can be incorporated in the project database and unnecessary duplication of earlier work is avoided. During the actual survey, it is necessary to ensure that the quality of results is continuously monitored and that data are processed and incorporated into the project database with minimum delay so that feedback from earlier stages of the survey can be used to guide decisions at subsequent stages. It will be necessary to apply the most up-to-date methods of data acquisition and management during the survey to ensure the quality and compatibility of the results. The ultimate aim of all procedures is to ensure that the survey is comprehensive, that the funds available are used effectively, and that all the data acquired during the survey can be placed at the disposal of the team responsible for design and construction as soon as possible.

3.3 Procedure for Carrying Out an Engineering-Geological Survey

The steps in the procedure for carrying out an engineering-geological survey for dam construction depend on the geological circumstances, the scale of the project and the funds available. For these reasons, the procedures used will vary from one case to another. An overview of the most commonly applied methods and techniques will serve as a guide for those involved in the different types of specialist work carried out at separate stages of a survey. The functions of the engineering-geological survey at each particular stage are summarized schematically in the flow chart below.



This chart should be used as a general guide. The sequence of activities involved in surveys at different types of site may vary. At each stage, however, the results obtained should be assessed to enable feedback. This feedback is particularly important at steps 5, 6 and 7, when the results must be compared with the scenario predicted by the working hypothesis. The construction procedures and design may require modification in the light of the survey results. If necessary, steps will be taken to find the solution to unforeseen problems.

3.3.1 Choice of a Survey Method

The choice of an engineering-geological survey method is governed by the following basic factors:

a) The information already available and the state of any existing survey work. These determine the level of detail required in the survey.
b) The intended use of the construction site. The choice of methods used and the extent of a survey for the body of a dam will be different from those used for ancillary plant and communications because different engineering-geological criteria apply. Linear constructions (diversion tunnels and roads) and areally extensive works (pits and quarries for extraction of construction materials, investigations in the backwater area of a dam) have different survey requirements.

c) Natural conditions at the site under investigation. Three categories of engineering-geological complexity can be distinguished based on geographical conditions, topography, geological conditions, hydrological and climatic conditions, and the relations to the surrounding environment (Tab. 3.3.1). And
d) External conditions. These include the difficulty of access to the site, the limitations on time available for the survey and construction, financial constraints set by the budget, the technical capacity of the survey organizations including their equipment and the expertise available and last, but not least, the level of technical complexity of the planned construction. Four categories of complexity can be distinguished (Tab. 3.3.2).

Table 3.3.1: Categories of areas based on the complexity of engineering-geological conditions

Category of complexity of area	Engineering-geological characteristics of the area (geomorphology, geology, hydrogeology, geodynamic processes)	Additional criteria required in reservoir area
A Simple engineering-geological conditions	Areas with simple geological, geomorphological and hydrogeological conditions; geodynamic phenomena are not present; sedimentary or magmatic rocks are homogeneous and strong without structural complications	Water seepage, failure of banks and changes in the area of submergence do not threaten the safety of the construction site and technical measures are not required
B Moderately complicated engineering-geological conditions	Areas underlain by a combination of sedimentary and magmatic rocks or sedimentary and metamorphic rocks with simple contacts. Geodynamic processes like slope failures, karst phenomena, etc., are little developed; groundwater forms separate horizons in different lithological units, its chemistry varies	Some problems affect the area of construction, but these can be safely dealt with using conventional technical measures
C Complicated engineering-geological conditions	Areas with complicated geological and geomorphological conditions; relationship between different rock units is structurally and geometrically complicated, unstable tectonic conditions (seismicity), geodynamic processes widespread; hydrogeology of different units is complicated	Significant problems affect the area of construction, and technically and economically demanding measures are required to guarantee safety

The first stage of preparation for an engineering-geological survey will always be one of consultation between the client, the designer and the engineering geologist to discuss the external constraints and the methods that will be used in constructing the chosen design of dam and infrastructure. These determine the loads that will be placed on the construction site and the surrounding environment, and the budget required to achieve a technically acceptable solution. The engineering geologist responsible for the survey can then plan the methods which

Table 3.3.2: Categories of complexity of structure based on technical characteristics

Type of dam				
	Dams constructed with natural construction materials (earth-fill, rock-fill, combined, earth-rock)		Dams constructed of man-made building products (concrete, arch, gravity and multiple) and special dams (metal, prefabricated sections, etc.)	
Category of complexity	**Type of subsoil**			
	Rock	Semi-rock and earth	Rock	Semi-rock and earth
	Maximum height of dam [m]			
I	>100	>50	>100	>25
II	50–100	25–50	50–100	20–25
III	20–50	15–25	20–50	10–20
IV	<20	<15	<20	<10

will be used for the survey, in particular the number, depth and layout of drill holes and excavations, and the type and number of field and laboratory tests.

The design and scope of the survey work must be such that the geological conditions and geotechnical properties of the rock mass in question are progressively defined in sufficient detail to enable the project to advance from one technical level to the next. The outcome should be a safe and economic water-retaining structure built to respect the sustainability of the surrounding environment and its ecology. The comprehensive work involved in the engineering-geological survey for dams and their associated infrastructure entails geological observations and mapping, stripping of topsoil, drilling, excavation of pits, trenches and adits, sampling, additional observations and tests in boreholes, field tests, petrographic examination and laboratory analysis. All of these types of work are integral parts of a comprehensive survey. The general order in the list above indicates the sequence in which the various procedures are used but not necessarily their relative importance. Different methods will be of greater or lesser use depending on the geological circumstances so the engineering geologist will plan accordingly, bearing in mind the questions that must be answered.

3.3.2 The Significance of the Engineering-Geological Survey in Preparatory Work and Construction

The geological conditions in an area where the construction of a dam and ancillary facilities have been planned are often complicated. This means that a professional and systematic investigation of the site must be carried out before the plans for construction are approved. A full survey requires a comprehensive range of engineering and related expertise. Many years of experience, sometimes dearly acquired, show that a limited number of boreholes and a few trial excavations cannot substitute for the knowledge of an experienced engineering geologist who will make an appraisal of the site based on a wider understanding of the regional and local geology and geomorphology, and on the problems that have arisen in analogous situations. Using this background knowledge, inferences can be drawn concerning the geological structure and superficial processes responsible for the local relief so that a rational and effective plan for drilling, excavation and stripping work can be proposed, and an appropriate programme of sampling and testing on site and in the laboratory can be recommended.

The full responsibility for ensuring the thoroughness and accuracy of an engineering-geological survey falls upon the engineering geologist who must collate and interpret all the information and test measurements that relate to the design of the project so that the results of the survey are both accurate and intelligible to the designer and others involved in making technical decisions. Therefore, in addition to having a good background in geology, especially structural and regional geology, geomorphology, petrography, and hydrogeology, it is most important that the engineering geologist is also trained in engineering and related disciplines, in particular geophysics, soil and rock mechanics, geochemistry, mineralogy, seismology and foundation engineering. Obviously, the wide scope of these fields of engineering means that it is impossible for an engineering geologist to be familiar with all the technical details. However, the engineering geologist should know sufficient about the principle involved that, when problems arise, the appropriate specialist can be called in to carry out the necessary tests and interpret the results.

An engineering geologist must have a good technical background but it is equally important to understand the natural context of the project in order to visualize the completed work and to understand the impact it will have on the surrounding environment, especially any adverse geological processes that might be triggered by the construction of the dam and its peripheral facilities. The engineering geologist must therefore grasp the engineering concepts on which the design of a dam is based so that the engineers responsible for the design and construction can be warned about the technical problems and risks that are likely to arise as the project proceeds because of geological factors specific to the chosen site.

An engineering-geological survey should provide all the information required by the designer and engineer to establish the technical feasibility of the project. In the first stage, it is usually sufficient to carry out an analysis of the existing archive of maps and reports, combined with field reconnaissance of the geology and an assessment of the topography of the site so that alternatives for the profile of the dam can be considered and the optimum solution can be chosen. Rectified satellite images and aerial photographs can be used to construct digital models of the terrain and preliminary geophysical measurements can be carried out. Later, this information will be amplified by observations and measurements obtained directly from exploratory drilling and excavation and by the wider application of geophysical methods. Geotechnical tests in the field and in the laboratory are then planned in relation to the geological composition and structure of the site, the results of which then provide the technical foundation for the design and construction of the dam in relation to the underlying geology. During dam construction, observation and description of new exposures and excavations continues progressively so that previously unrecognized complications in the geology of the site can be identified and the necessary technical adjustments can be made. Continuous revision of the geological database is important for the following reasons:

- It ensures that the engineering structure is based on the best available geological information, and enables technical adjustments to be made as the construction of the dam progresses;

- The observations and experience obtained during one project enable more effective planning of subsequent construction projects in analogous natural conditions; and
- The data obtained from a valuable addition to the bank of regional and local geological knowledge.

In the case of hydraulic structures it is also essential to carry out periodical surveys and systematic measurements to monitor the performance of a dam and related facilities after the work of construction is complete and the dam and its reservoir have been commissioned. This will draw attention to any weaknesses in the design and operation of the structure and allow remedial measures to be taken in time to prevent any serious failures. This information will also be fed back into the design criteria used for future projects in similar geological settings.

3.3.3 Basic Strategy for an Engineering-Geological Survey

The strategy employed for the engineering-geological survey of a dam site should lead to a progressively more detailed knowledge of the underlying geology and the factors governing the design of the planned water-retaining structure. In the wider area defined in relation to water-management interests, it is usual to begin with a regional geological survey. At this stage, it is still possible to change the specified location of the dam within the chosen area so that conditions for its construction will be optimal. Once the regional geological survey is completed and the final choice of the dam site has been made, the detailed engineering-geological survey will begin.

The regional geological survey will provide the following important information:

- The definition of the regional stratigraphy, the location of the main rock complexes and the positions of the geological contacts between them;
- An assessment of the tectonic history of the area and how this will affect the technical aspects of the building project;
- An analysis of the geomorphology of the area in relation to the geological history, the regional stratigraphy and the structure and the way in which these affect the action of ground and surface waters, giving special attention to the possibilities of water seepage into neighbouring basins;
- An evaluation of the existing groundwater regime;
- An assessment of the occurrence of mineral raw materials, both with respect to their use as construction materials during construction and operation of the planned dam, and with respect to their protection in case of a water-management interest;
- An evaluation of active and potentially active geodynamic processes, such as erosion, abrasion, suffusion, weathering processes, sedimentation, development of karst phenomena, creep movements, landslides, earth flow, fault movements and seismic hazards, rock falls, volcanic action, etc.;
- An assessment of the ways in which geodynamic processes could affect the site during construction and after commissioning of the dam and ancillary facilities;

- An evaluation of how human action will change currently active geodynamic processes, the state of stress in the rock mass and the groundwater regime and whether remedial measures or other technical procedures will be required to stabilize conditions at the construction site; and
- An assessment of the impact that the construction and operation of the dam and its facilities will have on the environment.

The time and expense involved in the preparation of a regional study will depend directly on the quality of the geological and other information available and the complexity of the planned construction. To fulfil the defined goals of the regional study, it will be necessary to:

- Study and evaluate archived geological documents, such as geological maps, sections, descriptions of exploratory workings, and reports relating to previous topographic, geological and hydrogeological surveys in the area, etc.;
- Interpret satellite and aerial images and aerial geophysical maps (if available) in terms of engineering geology;
- Conduct field reconnaissance of selected areas identified as important on the basis of the information gathered during the above evaluation and confirm or re-evaluate the geological information on them; and
- Map the areas where information on the underlying geology is absent so that geological sections can be constructed and the details of the underlying geology required for construction purposes can be supplied.

The regional study should record all the observations made during this stage of work. Another important task is to identify problems that could arise and draw up a list of questions that should be answered during the detailed engineering-geological survey that will follow. The regional geological survey of the area of Dalešice PSHEP can be used as an example. The existing geological map of the CR at a scale of 1:200 000 served as the basic source of information on the area. The geology was verified by field reconnaissance and a geological section along the planned profile of the future dam of the Dalešice reservoir was constructed (Fig. 3.3.1).

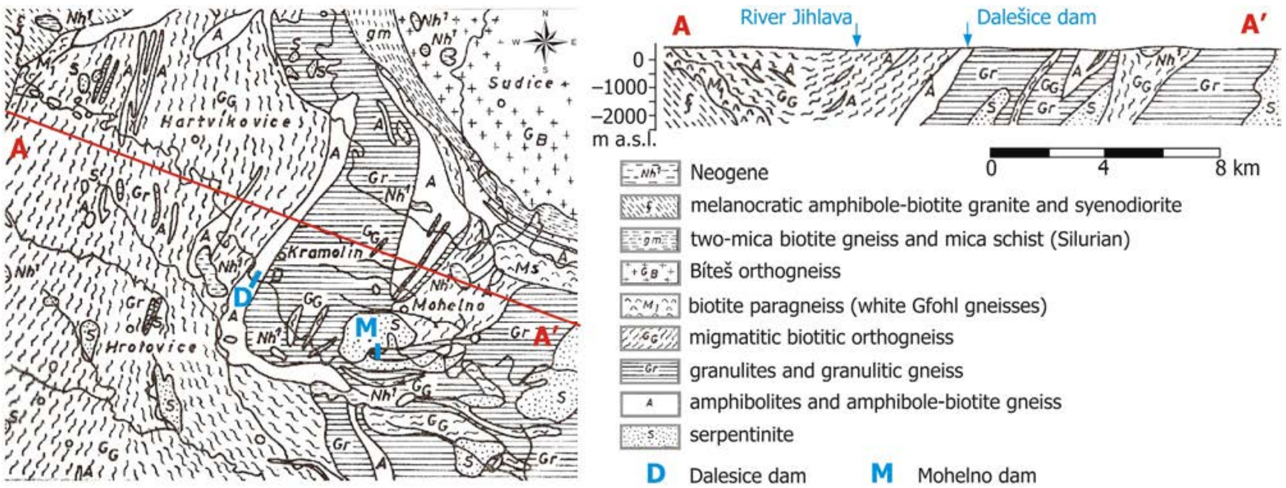


Fig. 3.3.1 Extract from the 1:200 000 scale map sheet showing the geology in the area of the Dalešice PSHEP. The red line A-A' marks the position of the geological cross section through the dam site shown on the upper right. The geological relations depicted in the section were verified by making field trips across the site

In countries with a well-developed geological survey, the first stage of an engineering-geological survey for a dam will be based on the reinterpretation of an existing regional model of the geological structure and tectonics. In less-developed countries, the situation is often such that the basic information required for an effective preliminary geological evaluation of the project area is lacking. This makes it more difficult to guarantee the successful progress of the engineering-geological survey.

An orientation survey is usually carried out using the limited amounts of available input data. This will usually be based on data obtained from satellite and aerial images, combined with geological maps at small scales, and topographic maps at small and sometimes larger scales. These sources of information will usually enable an interpretation of the tectonic structure of the area of interest. An example of an orientation survey of this type was that carried out to select a suitable profile for a dam on the River Genal (Fig. 3.3.2).

The morphological analysis of the area of interest was based on a topographic map of scale 1:50 000. Analysis of the drainage pattern served as the basic tool for the identification of tectonic lineaments. Using the pattern of topographic contours, the morphology of slopes was also studied to detect abrupt changes indicative of underlying fault structures. The results of this analysis are given in Figure 3.3.2 that shows the pattern of lineaments. It is natural that the lineaments identified will also coincide with zones of weakness in the rock mass. This weakness can be caused by tectonic fracturing, but can also be due to stripes of softer rock or hydrothermal alteration. It can be seen that some of the identified lineaments correspond with faults cutting through the geology depicted on the map at 1:50 000 scale, though some do not.

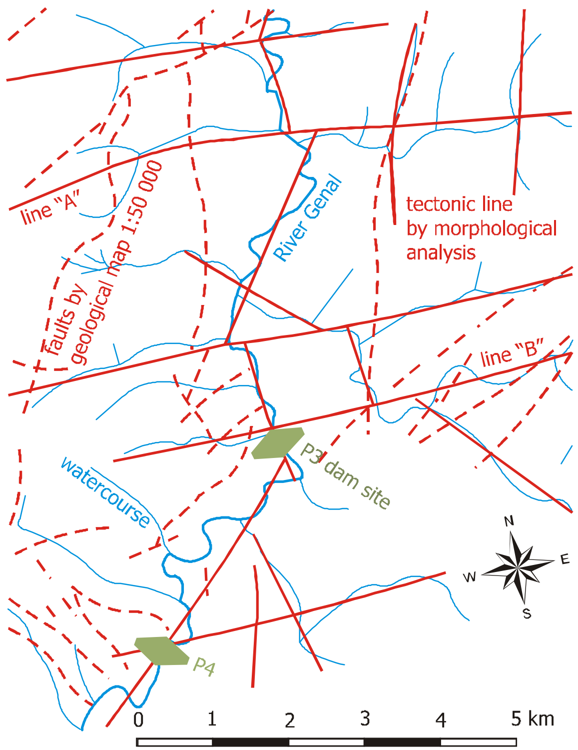


Fig. 3.3.2 Interpretation of the pattern of drainage in the River Genal basin in terms of the main structural lineaments

Perhaps the most significant difference between the pattern of lineaments interpreted from the topographic map and the geology shown at 1:50 000 scale is the E–W lineament crossing the central part of the studied area. However, the morphological analysis of sites chosen for the dam profiles does clearly show that an important topographic lineament lies in this direction in the central part of the area. This direction is also significant because the River Genal always changes its direction of flow between two such lines. Between these two belts the river flows from north to south. The general direction of the flow of the Genal is NE–SW, or NNE–SSW (see belts south of P4 and north of P3). Another feature governed by these E–W lineaments is the character of the flow of the Genal. North of the most significant E–W lineament (line A), the Genal has an obviously meandering course. South of this line, its course is much straighter. Moreover, between lines A and B the gradient of the Genal changes distinctly. To the north of lineament A it drops a vertical interval of 10 metres over 300 to 600 metres of its course, whereas between these lineaments the same descent takes place over 700 to 1,200 metres. It appears from this that the rocks that crop out between lineaments A and B are more resistant to erosion. In addition, this area means that the flow of the Genal is slowed. Hence, the Genal is forced to take a meandering course in the northern part of the terrain, even though this is the upstream part of the course of this river. Such meanders are sometimes described as super induced (or truncated) meanders because they are governed by the tectonic structure of the terrain through which the river flows.

Based on the morphological analysis of the Genal drainage, it was possible to predict that the geological structure of the area is complicated by a number of tectonic lines and zones. Geophysical measurements later confirmed this prediction. It turned out that the individual tectonic lineaments showed a strong correlation with the geophysical anomalies detected in the survey.

The survey carried out on behalf of the Centro Cuba PSHEP is a good example of the detailed stage of an engineering-geological survey. In this case the geology of the area was not known in great detail and there was a division of opinion among the geologists who had carried out the preliminary investigation of the area. For this reason a critical approach was required. At the stage of the preliminary survey, the background information appeared to be sufficient to make an assessment of the four sites proposed for the pumped storage hydroelectric plant (Hrdý, *et al.*, 1981). However, by the time a detailed geological survey was under way to provide the information required for the engineering design of the construction, it became evident that it was impossible to create the necessary engineering-geological and geotechnical models of the area of interest without answering some fundamental questions about the underlying geology that would enable the differences of opinion between the classical geologists and the engineering geologists to be resolved.

The detailed field survey was complicated by the fact that the area is dissected mountainous terrain covered by dense tropical vegetation. This made access to the area very difficult. The topographic base maps were also inaccurate and some basic questions relating to the structure and geology of the region had therefore not been properly answered before the technical details of the contract for the engineering-geological survey had been drawn up. Because the difficulty of access to the area had been underestimated by the Cuban contractor, the survey for an engineering design, originally scheduled to take 16 months, eventually took one year longer. One positive effect of this was that time was made available to investigate some basic aspects of the geology of the area that were still not understood. In this circumstance, it was not possible to follow a strictly logical sequence in the survey procedure. For example, under normal circumstances, a preliminary study of the area would be carried out remotely using stereographic pairs of aerial photographs. These should have been ordered before starting work on the ground, and the evaluation would already have been carried out in the Czech Republic because, in Cuba, the technical facilities for this were not available. In this case, the study of the aerial photographs was not successfully arranged until a year after the start of the survey. By this time the area of interest had already been geologically mapped at a scale of 1:5 000. It is important to bear in mind that in some countries, access to aerial photograph coverage is governed by issues of secrecy, especially detailed images at larger scales. Formerly, permission to work with large-scale maps and aerial photographs could only be obtained after months or even years of negotiation with the state authorities concerned. With the advent of public access to high definition satellite imagery through the Internet, this level of secrecy has become meaningless, but difficulties can still be encountered.

In the Centro Cuba PSHEP project, the Czech team were able to interpret the aerial photographs and produce a synthesis of the tectonic structure of the area which confirmed the conclusions reached on the basis of the mapping carried out on the ground by the engineering geologists. In addition, the interpretation of the aerial photographs drew attention to some important lineaments that had not been recognized during the ground survey. In order to verify these, it was necessary to gain access to terrain that was densely overgrown with tropical vegetation so that many more kilometres of profiles had to be surveyed in order to obtain the complete engineering-geological coverage required for the project. The tectonic framework of the area which was produced by this comprehensive geological survey is shown in Figure 3.3.3.

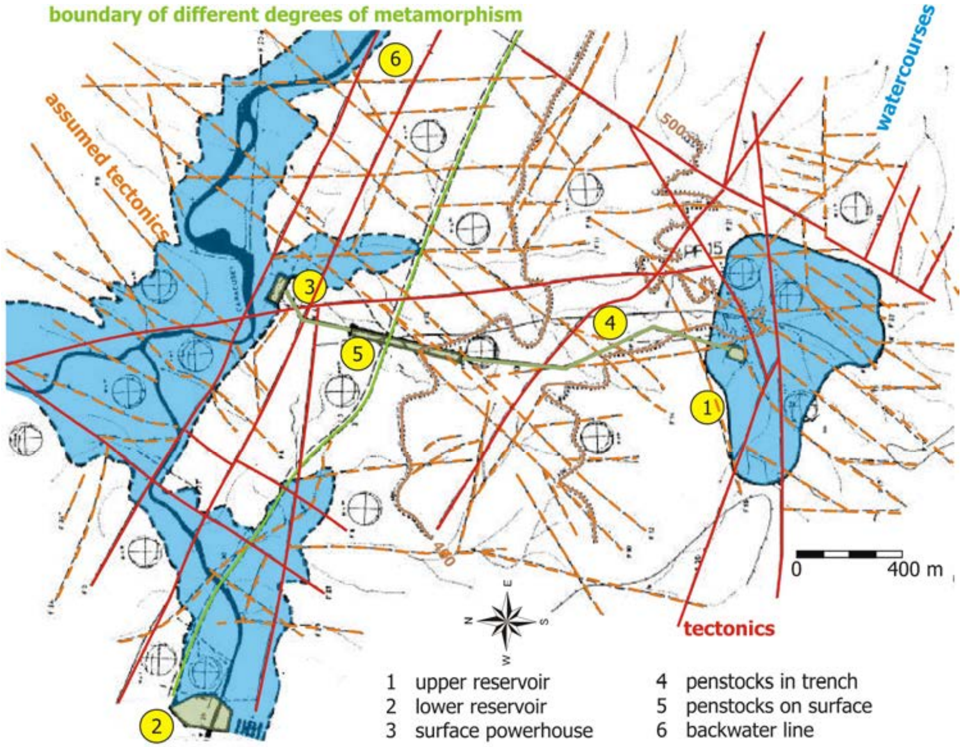


Figure 3.3.3 Map showing the drainage and topography of the area surrounding the reservoirs of the Centro Cuba PSHEP with structural interpretation superimposed. Solid lines are observed faults and the dashed orange lines are inferred faults

While the regional survey is designed to help in selecting the optimal position for a dam, the detailed survey is carried out in order to understand the engineering-geological conditions at the selected site. The main aim is to delineate all the zones of weakness and instability that occur in the area where the planned construction will take place so that a detailed structural and geological model of the site can be created. This will be constructed using observations of all the geological features that affect the planned construction. The model will incorporate all the geological data acquired by observation and measurement of rocks in surface exposures, trenches, pits, tunnels, boreholes, etc. This means that the structural-geological model is pinned by direct observation and testing of the underlying rocks. These observations are also supplemented by valuable information obtained from geophysical measurements of resistivity and seismic velocity along surveyed profiles and by the application of other geophysical and logging methods. The use of geophysics and down-hole logging enable the properties of larger volumes of the underlying rock mass to be explored, thus extending the knowledge gained by direct observation of surface exposures and excavations. The engineering-geological database is also improved by extrapolating the results of preliminary geotechnical tests to the whole rock mass. The progressive refinement of the geological model of the planned construction site in this way enables the strategy of sampling for laboratory and field tests to develop rationally as the survey work advances.

The engineering-geological maps of the site produced using the results of the preliminary and detailed surveys show the spatial relationship of all the observations and measurements that have been gathered. The first stage is to create a factographic map that is an objective summary of all the data and information obtained. The second stage is to construct an interpretative map on which the geology, structure and engineering properties of the site are used to create an integrated model constrained by the original observations. Depending on the amount and type of information available, the map can be compiled in one or more layers that contain engineering-geological, hydrogeological or other information that will govern the procedures used in construction and operation of the dam and its ancillary facilities. To guarantee that the engineering-geological, hydrogeological and geotechnical information gathered during a detailed engineering-geological survey is adequate to ensure the technical success of a dam construction project, the survey should generate the following outputs:

a) Detailed engineering-geological mapping of the future reservoir area using topographic base maps at a scale of 1:5 000 or 1:10 000. Mapping of the dam profile itself and the important structures in the area of construction should be carried out at a scale of 1:1 000. An appropriate example of a detailed engineering-geological map at a scale of 1:1 000 is that covering the Dalešice dam site, including the diversion tunnels and the associated pumped storage hydroelectric plant. The complexity of the geological structure at this site had a fundamental influence on the choice of the type of dam construction used. Instead of the concrete arch dam that was originally planned for this site, a rock-fill gravity dam with a clay core was constructed (Fig. 3.3.4). The basic engineering-geological map summarizes data in six important categories:

- Description of geomorphological features within the area of the dam project, namely the pattern of the drainage and the gradients of the rivers, the character of the relief showing altitudes above datum and differences in relief, gradients of slopes and their state of dissection, the classification of the area with regard to different landforms, the relationship between macro- and mesoforms of relief, and the evolution of the individual features (erosion of gorges, distribution, type and evolution of slope failures and potential instabilities, morphology of floodplains, chronology and definition of river terraces, microgeomorphological and cryogenic features, etc.). Much of this detailed analysis, and the geomorphological classification of the terrain can be carried out effectively using aerial photographs and satellite images;
- Geological features of the superficial deposits within the area. Particular attention will be given to identifying the different types of Quaternary cover, such as alluvial terraces, colluvial and eluvial deposits, glacial till, moraine, loess, periglacial solifluction deposits, talus fans and other mass flow phenomena and the way in which instabilities and slope failures are related to them. By this stage much of the information will be gathered by carrying out field trips;

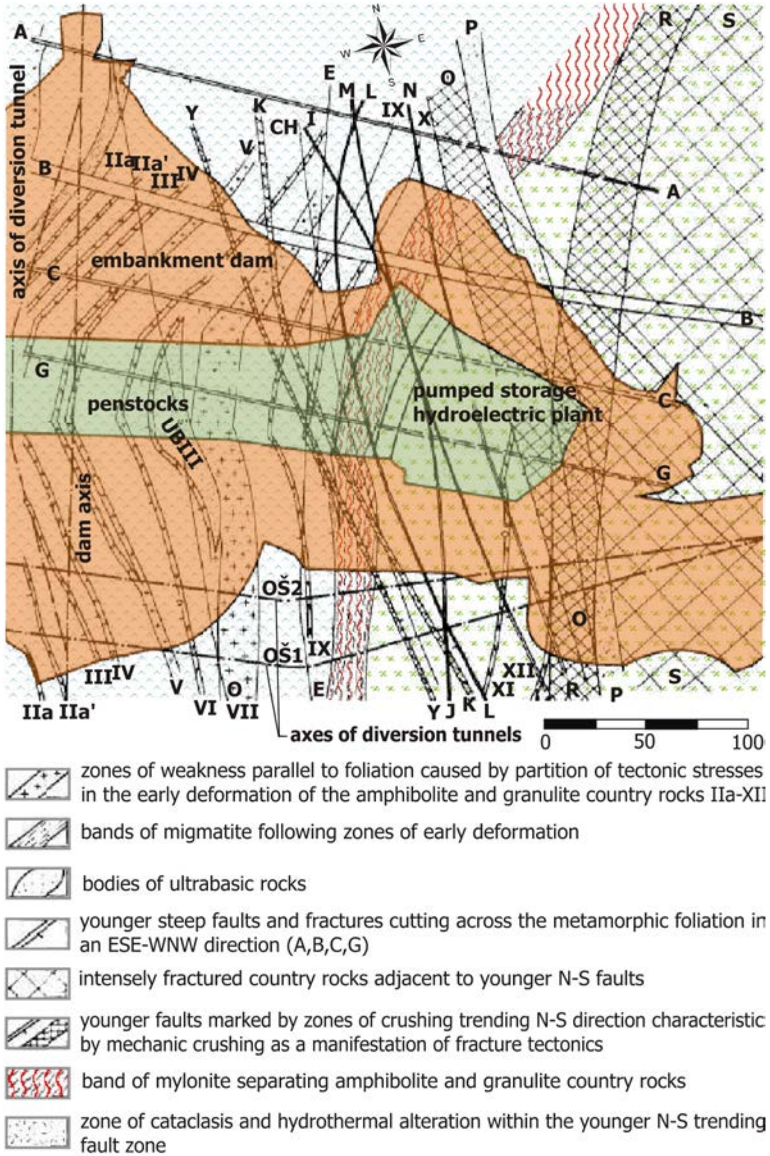


Figure 3.3.4 Example of a detailed geological map, originally at a scale of 1:1 000, showing the rocks and structures at the Dalešice dam site

- All exposures of the underlying bedrock are depicted and the geological contacts beneath the superficial cover will be projected to show the different geological formations underlying the area. Particular attention will be given to zones of weathering, mineralized zones and related hydrothermal alteration that give rise to weakness and permeability;
- The geological structure of the area is analyzed in terms of its tectonic history using the pattern of folds, faults, foliation, lineation and fracturing in the rock mass. Particular attention is given to all discontinuities in the rocks that result in mechanical weakness and increasing/decreasing permeability;
- The maps depicting the distribution of superficial deposits and the underlying rocks are used to locate the samples selected for petrographic and geotechnical investigation. A database containing information about the different types of rocks and soils and their mineralogical composition and geotechnical properties is created. This will include information about the anisotropy of the different rock formations, their permeability and the state of stress within them; and
- Information about the hydrogeology and the groundwater regime in the area is also compiled. This includes the depth of the groundwater table and its seasonal fluctuation, directions of groundwater flow, locations of springs, delineation of areas where seepage occurs and the determination of the physical-chemical properties of the groundwater so that its source, chemical reactivity and seasonal variations can be assessed. The engineering-geological map, on which the above information is compiled, together with the necessary cross-sections, is the primary output of the engineering-geological survey on which subsequent technical decisions depend.

b) The second major output of the survey is the construction of the engineering-geological model of the defined block of ground on which the dam and ancillary facilities will stand. Stereoscopic images of the terrain can be used for this purpose, but more recently it has become possible to construct digital models of the terrain using rectified satellite images. In cases where the geology is complicated and it is vital that the dam is located precisely in relation to the underlying structures, it is necessary to build a three-dimensional working model (Fig. 3.3.5). This is an essential tool that allows the geologists and engineers to visualize the progress of the project in relation to the underlying geological structure and plan additional survey work in areas where problems are encountered. The GIS environment is now available that enables a digital model to be created in virtual space. Such models can be explored by rotation and translation of the viewpoint and constantly remind the geologists and engineers how the main zones of weakness are distributed in the rock mass below and around the construction site and how far these have been explored by drilling and excavation.

Fig. 3.3.5 Perspex model of the Dalešice dam site (a photo by O. Horský - 1969)

c) The geophysical methods are used to develop a more detailed and objective understanding of the geological structure. Most frequently, resistivity and seismic

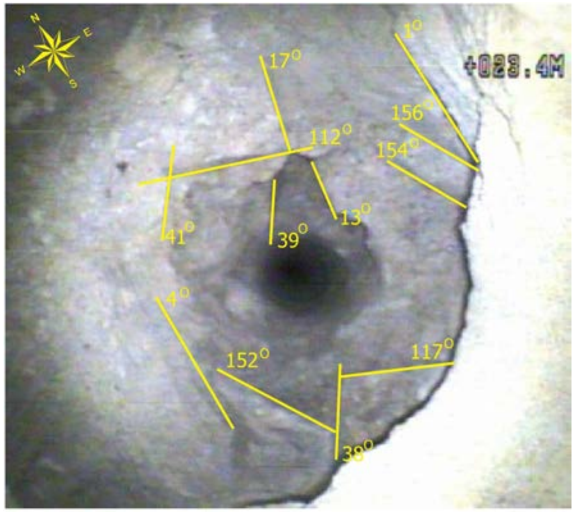


Fig. 3.3.6 TV image of a cavity with planes of weakness and their azimuths projected in yellow

methods are used, but electromagnetic, magnetic, radiometric, thermal and gravimetric methods and ground-penetrating radar can also be useful depending on the circumstances. In addition to the data gathered during geological and hydrogeological mapping, these methods provide information about the physical and mechanical properties of the underlying rock mass.

d) Boreholes and man-made exposures, including exploratory pits, trenches, tunnels, cuts and other excavations are an important part of the engineering-geological survey for dams. Exploratory drilling provides core for description and geotechnical testing and excavations provide an opportunity for direct observation of the geology, as well as sampling of the walls of pits, trenches and tunnels. The information obtained in this way forms an essential component of the engineering-geological model and the geotechnical database. Drill holes and excavations provide many of the samples required to produce a detailed classification of the rocks and soils. They also enable measurement of the principal structural features such as bedding, cleavage, faults and fractures, and zones weakened by shearing and hydrothermal alteration. Measurements of the physical and mechanical properties of the rocks can be made ***in situ*** and on samples of drill core and rock taken from the exploratory workings to the laboratory. The range of information available is also increased by down-hole logging and by using a probe equipped with a digital camera, and by photographic and video documentation of the exposures in boreholes, tunnels and in stripped areas on the surface. It is also possible to determine the orientation of structures within the holes by using special down-hole tools equipped with a digital camera and compass (Fig. 3.3.6). The exploratory cut made at the foot of the slope on the right side of the Dalešice dam site is a good example of an artificial exposure created to enable direct observation of the rocks and structures along the profile chosen for construction of the dam. The cut in the slope was made along the line of an existing forest path coinciding with the profile chosen for the dam. The aim was to expose the rocks and the geological structures along the part of the

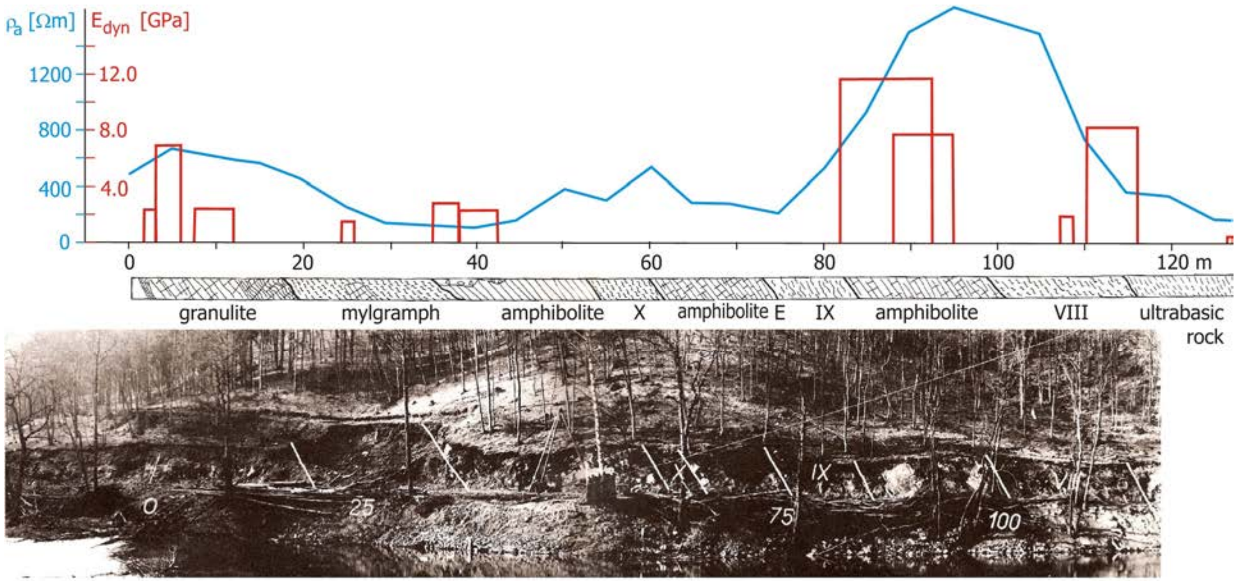


Fig. 3.3.7 View of the cutting made along the foot of the right bank at Dalešice showing details of the geological profile and the measurements of the electrical and mechanical properties of the rocks (a photo by O. Horský - 1969)

profile of the future dam that would be most heavily loaded by the structure. In addition to routine geological observations, a shallow seismic refraction and resistivity measurements were carried out along the line of the cut. The seismic measurements were used to calculate the dynamic modulus of elasticity of the rocks. The artificial cut thus enabled the interpretation of the geological structure of the rock mass to be verified and the attitudes of the main planes of tectonic disturbance in the area of the dam foundation to be measured (Fig. 3.3.7). And

e) Progressively, all the information obtained in the successive stages of the survey is incorporated into the engineering-geological map thus enabling the integrated three-dimensional geological model to be built up. This procedure allows the geologists and engineers to continuously review the data acquired so that the investigations can proceed in a logical order and to ensure that the information is comprehensive and of adequate technical quality. The engineering-geological model provides the basis for a structural analysis of the rocks underlying the dam and ancillary facilities, but the value of this analysis will always depend on the accuracy with which the different structural features have been measured and incorporated in the model.

The results of each stage of the survey work are compiled in separate reports for evaluation. In surveys for larger dams and ancillary projects it is usual to prepare a final summary report that describes the scope of each stage of the engineering-geological survey, the problems identified and the solutions proposed.

3.4 The Planned Stages of an Engineering-Geological Survey

The stages of an engineering-geological survey in preparation for the dam construction, as in other major civil engineering projects, are governed by the level of information required for the completion of each stage of the planned design. The successful initiation of each new stage of dam construction depends on the satisfactory completion of the previous stage. Because of this interdependence, the survey must proceed stage by stage so that the results of each stage can be used as a basis for carrying forward the engineering-geological survey to the next level required to support the technical decisions that will enable the design and dam construction to proceed safely and efficiently.

The step-by-step rule governs the survey procedure used in all technically developed countries unless there are special circumstances that permit stages to be merged so that more rapid progress can be made from one stage to another (Tab. 3.4.1).

The design stages corresponding to the progressive survey stages are as follows:

- A preliminary study of the river drainage and the surrounding catchment designated for the dam project so that the various sites suitable for the construction of dams or power works can be identified;
- Project design and economic feasibility aimed at assessing the engineering-geological conditions at the different sites and the material and financial resources required for construction in the different scenarios. Decisions regarding the financial and technical feasibility of the project will be taken on the basis of this analysis and the most suitable site will be chosen;

- Preliminary and master engineering projects will then be proposed and their technical viability assessed so that a final decision on the selection of the construction site can be taken. This will take account of all the details of the proposed design, including the technical parameters of the dam itself, the layout of the ancillary facilities and the area of the reservoir. The engineering project will incorporate measures for preventing water seepage, and for ensuring the stability of slopes, and of the excavations and the dam construction itself; the engineering-geological survey on which the technical decisions summarized above are based can be divided into two separate sub-stages, namely:
  - A survey to enable the final selection of the most suitable site for the dam if the choice has not been determined already by the stage of project design; and
  - A survey of the selected dam site; and

Table 3.4.1: The stages of the engineering design and EG survey of dam projects

Design stages						
Former USSR	komplexnoje ispolzovanije reky	technikoekonomi-cheskoje obosnovanije	technicheskij projekt	rabochije cherteji	vo vremja strojitelstva	posle strojitelstva
USA Japan	preliminary study	prefeasibility study	feasibility study	detailed design	during construction	after completion
Argentina	inventario	Prefactibilidad anteproyecto somero	factibilidad ante-proyecto avanzado	proyecto	constructiva	operativa auscultación
Cuba	subetapa de tarea de proyección	tarea de proyección	proyecto técnico	Proyecto ejecutivo	construcción	operación auscultación
India		prefeasibility study	feasibility study	detailed design	during construction	after completion
Spain	previabilidad	viabilidad	ante proyecto	proyecto	construcción	puesta en servicio
CR	multipurpose utilization of river (preliminary study)	project design and economic feasibility (investment task)	preliminary and master engineering projects	detailed (implementation project)	construction	work operation, monitoring
Engineering-geological survey stages						
CR	inventory survey	orientation, and preliminary surveys	preliminary to detailed survey	detailed survey	additional survey and sequence	observation on dams
India	reconnaissance		preliminary investigation	detailed investigation	construction	after completion

- Detailed project (implementation of the construction project), during which the engineering geologists work closely with the engineers monitoring and constraining details of the excavations and foundations for the dam and individual ancillary structures.

In the recent past, the stages of engineering-geological surveys carried out in the Czech Republic were governed by “Guideline No. 1” issued by the Czech Geological Bureau in 1975.

This Guideline divides the survey into orientation, preliminary, detailed and supplementary stages followed by the engineering-geological monitoring of the construction process. In other countries, it is also customary to link the individual stages of an engineering-geological survey systematically to the process of design and construction. The current practice in a number of countries is shown in Table. 3.4.1.

Table 3.4.2: Division of survey into stages

Category of complexity based on technical characteristics Table 3.3.2	**II to III**	**I**	**II to III**	**IV**	**IV**
Category of complexity based on engineering-geological conditions (Tab. 3.3.1)	A	A B C	B C	A	B C
Recommended number of stages	2	3	3	1	2

The division of a project into design and survey stages using these principles generally takes place, but sometimes stages are merged for management or technical reasons. For example, in the case of reservoirs with dams less than 20 metres high founded on bedrock or those 10 to 15 metres high founded on unconsolidated rocks and soils, the stages of the engineering-geological survey can be merged if there are no geological complications (category A, Tab. 3.3.1). The recommended division of a survey into stages depending on the category of complexity of the engineering-geological conditions (Tab. 3.3.1) and on the category of technical complexity of the whole construction is given in Table 3.4.2.

The engineering-geological survey for the Centro Cuba PSHEP is an example of a dam project in which all the survey stages, with the exception of the orientation survey to identify a suitable site, were undertaken during four years of uninterrupted work. The completion of this demanding survey in such a short period of time was stipulated under the terms of the contract with the client. It was not the most economic solution. Completion according to the schedule required enormous expenditure and the deployment of a large workforce.

3.5 Technical Specifications and Plan of Survey Work

3.5.1 Technical Specifications

Technical specifications are an integral part of the contract for an engineering-geological survey. The client (building developer, designer, etc.) will have reviewed the technical requirements of the project, taking into account previously existing knowledge of the site and the results of any investigations that have been carried out. On this basis the client will identify the principal questions that are to be answered by the proposed engineering-geological survey. Due to the fact that the technical details of most projects are complex, especially at the more advanced stages of the design process, it is usual for an engineering geologist to cooperate with the client in formulating the principal questions that will be answered by the survey. For the reconnaissance study of alternative sites in the drainage basin and for the project design and analysis of economic feasibility, there are a number of basic questions that must be answered. These govern the selection of the most suitable site in terms of topography, hydrology and underlying geology in relation to the surrounding environment and what the project will cost. The technical specifications for the more advanced design stages should be based on the following considerations:

- The defined aims of the project, discussing the advantages and disadvantages of the possible alternative sites for the dam and the ancillary facilities associated with the water-retaining structure;
- Designation of the stages for which the engineering-geological survey is to be provided, taking into account the category of geological complexity of the area (Tab. 3.3.1) and the complexity of the construction (Tab. 3.3.2);
- Preliminary or definitive parameters of the construction work in relation to the chosen design, in particular:
  - The shape and type of dam and its dimensions in relation to the chosen site and the volume of construction materials required; and
  - The volume of backwater held in the reservoir and the anticipated maximum and minimum elevations of water level in the reservoir;
- Basic information about the methods used for construction, such as the dam fill, the concreting, water diversion during construction, requirements for drainage or sealing of construction pits, the type of foundation to be used for the structures, etc.;
- The identification of problems caused by particular conditions on the construction site or because of procedures used in the process of construction (e.g., the impact of blasting in a nearby stone quarry on the dam during construction and after the dam is completed, the risks posed by seismicity in the area, the changes that the dam will cause to the original environment, and the prediction of instabilities that could lead to serious slope failures in reservoirs, etc.);
- Prediction of any conflicts of interest, incurred investments and potential problems that construction and operation of the dam may cause to local infrastructure together with the requirements for engineering-geological solutions required; and
- Clarification of rights of access to the land and the various special regulations for protection of the landscape where these are in force, and the measures that must be adopted to avoid damage in such cases, etc.

In addition to the detailed review of the questions that the survey will address, a set of graphic appendices must be compiled as part of the technical specifications, especially:

- Maps at 1:25 000 to 1:50 000 scale showing the area affected by the designed construction;
- A topographic base map at a scale appropriate to the stage of the survey covering the reservoir area with the dam site and the surrounding area that will be affected during its construction;
- A detailed map of the topography of the dam site and the sites of ancillary structures at an appropriate scale; and
- Topographic sections through the main works showing the degree of detail appropriate to the stage of the survey.

At the preliminary stage of the project, a detailed topographic survey of the dam site and of all, or part of the backwater area will usually be required. The efficiency and technical reliability of the geological survey work depends on the accuracy of the topographic surveys that are carried out at this stage.

3.5.2 Designing the Geological Survey Work

The plan for each stage of the geological survey of a dam site depends on a comprehensive review of the geological and related technical information that has been acquired during previous investigations. This ensures that full account can be taken of the results of previous work and duplication is avoided. The plan of survey work must address all the questions raised in the technical specifications at the level of detail required by the given stage of the survey. To fulfil these goals, it is necessary to provide:

a) A clear working hypothesis that explains the geological conditions in the area of interest, updated using the results from the preceding stage of the survey. If the preceding stage led to a substantial change in the working hypothesis, attention should be drawn to this. It is most important to classify the main types of rocks in terms of their mineralogical composition and texture and depict their spatial arrangement. Units will be grouped into sedimentary, igneous or metamorphic formations with definite compositions, facies and structures that govern their engineering-geological properties. The essential features of the tectonic structure must be described and attention will be drawn to the potential weaknesses that must be taken into account in designing the proposed dam. The depth of the water table must be estimated and the chemistry of the groundwater and the character of the groundwater regime established, taking account of differences in the permeability of individual types of rocks and soils, etc. And
b) A geotechnical model, which is based on geological analogy, using the available archive of information and the findings of the preceding stage of the survey. An important component of this model is the estimation of the physical and mechanical properties of the bedrock and the overburden sediments and soils.

As a general rule, the working hypothesis for the engineering-geological and hydrogeological conditions in the area of interest will be illustrated by maps and sections (e.g., dam profiles), and relevant data can also be presented in tables with a short explanatory text. This will be based largely on the available knowledge about the area of interest supplemented, whenever possible, by reconnaissance in the field. In those rare cases when projects are undertaken at sites where the geological conditions are relatively simple (category A), the working hypothesis can be explained in writing without detailed drawings.

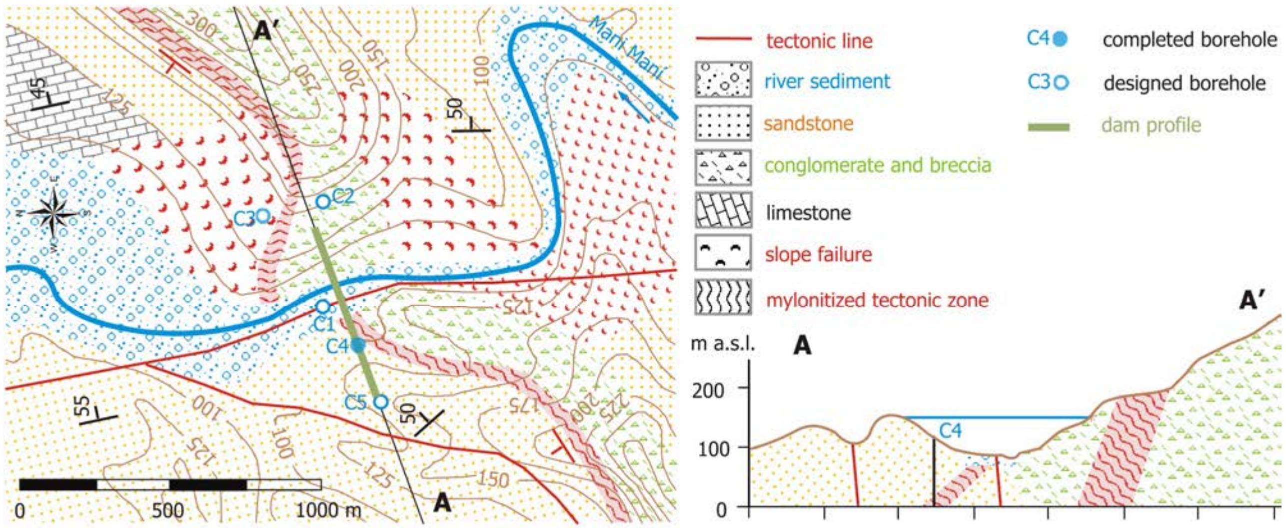


Fig. 3.5.1 Geological map and section of the San Miguel dam profile showing the location of planned boreholes

An example of a tentative working hypothesis is the profile compiled at a scale of 1:10 000 for the San

Miguel dam. This is shown in Figure 3.5.1. The existing information was tested and confirmed by a field trip made in the course of one working day. The material served as a basis for preparing a plan for more detailed survey work. A drill rig was already located on site and the designer needed information to complete the investment feasibility study, so a series of boreholes were set out immediately to verify the geological structure and test the mylonitized fracture zone that had been identified as a crucial structure in the working hypothesis.

The working hypothesis serves as the basis for proposals regarding the scope and type of survey work that will be necessary to carry the project to a more advanced stage. This must comply with the requirements for each of the designated design stages and must be adequate to ensure the technical integrity and safety of the project design. In making these proposals, the following rules should be observed:

- The 'step-by-step' principle governing survey work must be obeyed. It is not acceptable to merge separate stages of a survey if the work of the following stage depends on the results of the preceding stage, or if the engineering-geological conditions in the area are complicated (Tab. 3.3.1), or the category of technical complexity of the construction is I or II (Tab. 3.3.2); and
- The survey work must be comprehensive. The survey must guarantee the quality of information necessary to carry out the design and construction of the dam under the climatic, topographic, geological, and hydrogeological conditions prevailing. The information must be delivered on schedule so that the project can proceed safely and economically. The quality of the information will be assured by applying the best available knowledge of theory and practice to solve the problems faced in the conditions specific to that project.

The plan for the survey work should include the following:

a) Engineering-geological work:

- Definition of the scope of the engineering-geological mapping, the details of the methodology to be used and the procedure for compilation including the scales of the working maps and sections;
- Requirements for geodetic survey work, acquisition of topographic base maps, aerial or satellite images, etc.;
- Plans for the location, depths and number of exploratory boreholes and testing pits;
- Specification of the techniques to be used for drilling and excavation work in relation to the types of geological investigations to be carried out;
- Procedures for logging and describing the borehole core and pits so that all geological and technical information is recorded. Even though not all this information will be of direct use in completing the planned project, part of it may be useful in a different context;
- Anticipated plan of rock and soil sampling for petrographic, palaeontological or other analysis, including proposed sample locations;
- Specification of the geotechnical field tests, criteria for selecting the rocks to be tested and the anticipated number of samples;
- Criteria for selecting the sites from which soil and rock samples will be collected for geotechnical testing in the laboratory and from which water samples will be collected for chemical analysis and the predicted number and type of laboratory analyses required;
- Specification of the sites and methods to be used for field hydrogeological tests and measurements, such as measurement of the yield of springs, pumping tests, slug and indication tests, etc., and the scope required;

- Anticipated plans for additional use of drill holes, drill core and exposures produced by stripping, trenching, etc., and the measures that must be taken to prevent damage to them. If no further use is planned, the procedures for restoration will be specified;
- Plans for geophysical measurements that will be used to corroborate and extend the knowledge of the geological structure obtained from surface observation, geotechnical measurements, hydrogeological tests, drilling and stripping. The methods to be used will be specified together with the profiles along which measurements are to be made;
- Requirements for coordinating individual types of work to create a coherent schedule for completion of the planned survey; and
- Procedure to be used to process the data and report on the results of the planned survey.

b) Geophysical work:

- Brief summary of the objectives of the geophysical work to be carried out at the individual stages of the survey;
- Description of the different techniques to be used for the geophysical work and the required working procedures; and
- Specification of the methods used to process the data and the format in which the results are to be reported.

c) Geotechnical work:

- Laboratory tests:
    - o Brief summary of geotechnical tests to be made in the laboratory and their relevance to the engineering-geological aims of the project;
    - o Types of laboratory tests to be carried out and the required working procedures (transport of samples, etc.);
    - o Procedures for collection, storage and preparation of samples; and
    - o Methods used to process the data and the format in which the results are to be reported.
- Field tests:
    - o Brief summary of the scope and type of geotechnical tests carried out on the rock mass in the field;
    - o Methods of testing and working procedures;
    - o Technical preparation required for field tests and the optimum conditions for carrying them out; and
    - o Methods used to process the data and the format in which the results are to be reported.

d) Hydrogeological tests and measurements:

- Brief summary of the programme of hydrogeological tests and measurement in relation to the engineering-geological aims of the project;
- Types of tests and measurements and the required working procedures;
- Preparations required to make tests and measurements; and
- Procedures to be used for processing data from hydrogeological tests and measurements and the format in which the results are to be reported.

e) Laboratory testing and analysis of water:

- Brief summary of the type and number of laboratory tests to be carried out;

- Techniques required for collection, protection and transport of water samples to the laboratory and the time involved in transport;
- Methods for determination of unstable components at the site of collection or for their preservation; and
- Procedures to be used for processing the data obtained from hydrogeological laboratory tests and for reporting the results.

f) Other special tests and measurements, and technical work:

- Brief summary of the purpose of special technical work, tests and measurements in relation to the engineering-geological aims of the project;
- Types of tests to be carried out, and required working procedures;
- Conditions necessary to ensure that the tests are carried out successfully; and
- Procedures to be used to process the data obtained from special tests and measurement and the format for reporting the results.

g) Geodetic work:

- In projects covering a large area, it will be necessary to specify the requirements for surveying or resurveying of selected areas in order to produce topographic base maps of the scale and accuracy required to plan and manage the project effectively;
- In an atypical project, a separate plan is prepared for geodetic work, including procedures for cartographic compilation and the production of working copies, etc.; and
- Procedures for processing and storing survey data and the formats in which the results of the geodetic survey will be presented and used including GIS applications.

h) Specification of geological conditions for drilling and excavation work:

- Description of the anticipated geological profiles on which boreholes will be sited and along which excavation work will take place so that the necessary budget can be allocated and the appropriate rigs and machinery provided;
- Technical specifications for drilling work, including drill speed (revolutions/min), thrust, types of drill bits and diameters, lengths of casing and rods, flushing, and special problems that might occur during drilling (losses of drilling fluid, fall or jamming of tools, and other problems that might affect the efficiency of drilling and core recovery, etc.);
- Procedures for collection, stabilization, storage and description of core and other geological samples, including rocks, soils and waters that will be used for testing and evaluation of their properties;
- Geotechnical and physical methods used for direct measurement and description of exploratory workings; and
- Procedures to be used for processing and recording the measurements and observations obtained from drilling and excavation work and the formats in which they are to be reported.

i) Requirements for cooperation with other subjects. And

j) Conflicts of interest.

If, during survey work, an engineering geologist becomes aware of conflicts of interest that could obstruct or fundamentally affect the plan of survey work, this will be drawn to the attention of the client so that a solution can be found. If the problem identified leads to restrictions in the scope of the survey, changes in the plan or in the regulations governing the operation of the proposed dam, it will be necessary to find a solution by negotiation with all the interested parties. Previous experience has shown that, in many cases, conflicts of interest can be precluded by carrying out a comprehensive study.

The conflict of interest between the operator of the stone quarry in Bílčice and the future operator of the planned dam at Slezská Harta is a good example of a hydrogeological problem that had to be resolved. To do this, it was necessary to carry out a relatively extensive hydrogeological survey in order to assess whether water could leak from the reservoir into the expanded stone quarry. Figure 3.5.2 shows that, at the time of the survey, the quarry floor lay at an elevation of 505.0 m a.s.l. This was higher than the predicted maximum level of the backwater in the reservoir. However, the plan was to lower the quarry floor to an elevation of 485 m a.s.l. in order to increase the extraction of basalt. In this case, water could have leaked from the reservoir into the stone quarry; and for this reason the mining company was not permitted to increase the depth of the stone quarry as planned.

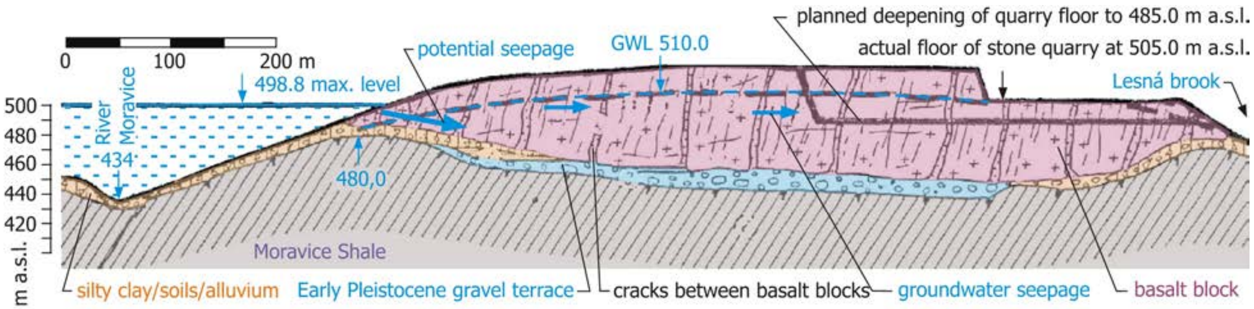


Fig. 3.5.2 Geological cross section along the right bank of the River Moravice at the position of the Slezská Harta dam showing the relationship between the water level in the dam, the water table (GWL) in the basalts and the planned deepening of the floor of the Bílčice quarry.

The resolution of conflicts of interest is a delicate matter. In many cases, the interest of the state overrides local or private interests. In such cases, conflicts are resolved by deciding in favour of the state and the project may proceed, in principle, without further complications. Because large dam projects often have far-reaching social and environmental implications, the protection of private property and land rights may become a fundamental issue. In such cases conflicts of interest are difficult to resolve because communities do not want to be displaced and traditional patterns of land use are disrupted. In such cases surveys may be stopped or limited until agreement between the interested parties can be reached. In ordinary circumstances, the private client and the government planning authorities will have done their best to resolve conflicts of interest so that the organization contracted to carry out the engineering-geological survey will not be faced with intractable problems. Nevertheless, in the reconnaissance stages of a survey, consideration must be given to potential conflicts of interest that emerge during evaluation of satellite and aerial images or as a result of aerial geophysical surveys. The plan of geological survey work must therefore take account of necessary liaison with the local community and interested organizations and lines of communication for this purpose must be established.

A very important part of the plan for an engineering-geological survey is the estimated budget required to carry out the work and the schedule for its completion. This component of the plan is absolutely crucial for the proper management of a contract.

The overview given above identifies the main tasks that must be incorporated in any plan for a comprehensive engineering-geological survey of a dam project. Other tasks may also be included in the plan depending on the natural conditions at the site and the specific technical requirements of the engineering design.

Nowadays, it has become the practice for a contracting authority to employ consultants to define the scope and the methods to be used in an engineering-geological survey and for companies to tender for the contract by submitting an appropriate budget for the proposed work. This system has definite disadvantages because the companies may be unable to bring the full benefits of their knowledge and experience to the project and the consultant may have overlooked the possibilities for using the most advanced equipment and procedures to carry out the survey. This contracting procedure can thus result in a technically inadequate survey because of failure to specify the scope and methods compatible with the geological circumstances and the design of the proposed dam. In this case the legal responsibility for breaches of contract and technical failures that could arise is not easy to apportion.

4 Engineering-Geological Mapping

4.1 Definition of the Tasks of Engineering-Geological Mapping

The design of hydrotechnical structures requires knowledge of the geological structure and geomorphology, as well as of the hydrogeological regime and the geodynamic processes acting in the area of interest. The engineering-geological map provides this fundamental information. An engineering-geological map is therefore the tool that allows evaluation of all the factors affecting the design of the dam and its ancillary structures. Engineering-geological mapping must therefore also be planned to allow for any detailed surveys that may be required for individual parts of the overall structure.

The choice of scale for the engineering-geological map depends on the complexity of the geological structure and the geomorphological conditions, and on the technical parameters governing the design and construction of the dam and ancillary facilities. The choice of scale also depends on the design stage. Engineering-geological maps can be divided into three main groups, depending on the scale of the topographic base maps on which the geological observations are compiled:

- Synoptic: 1:200 000 to 1:75 000;
- Basic: 1:75 000 to 1: 5 000; and
- Detailed: 1:5 000 to 1:1 000 and more.

Synoptic maps are usually compiled as a single sheet. Only in cases where natural conditions are complicated will a synoptic map of engineering-geological conditions and a map of zoning be compiled separately. In most countries, synoptic maps are used in order to carry out a preliminary evaluation of the engineering-geological conditions in the region chosen for a dam construction project, and such maps cover relatively large areas.

The basic and detailed maps consist of sets of several sheets. The most important of these are maps of engineering-geological conditions and engineering-geological zoning. Important geotechnical parameters such as permeability of rocks and soils, stability of slopes, abrasion, etc., that govern the design and construction of the dam will be compiled on auxiliary sheets. An important component of the engineering-geological documentation is a map summarizing the extent and type of fieldwork and laboratory investigations that will be carried out.

In geologically simple areas, it is acceptable procedure to compile all engineering-geological and hydrogeological data on one map sheet. In more complicated areas, it will be necessary to show hydrogeological data on a separate sheet so that the information is clearly legible.

The basic engineering-geological map is conventionally used for evaluating different proposals for the dam profile within the area delineated for the project, and for assessing the engineering-geological conditions affecting ancillary constructions such as long diversion channels, realignment of roads, etc. Usually, such maps will not cover areas greater than a few tens of square kilometres. Once the sites for construction of the dam and ancillary structures have been chosen, detailed engineering-geological maps are compiled for those specific areas of interest. These maps show the results of engineering-geological mapping and test work carried out specifically to prepare for the construction of the individual facilities. Such maps will usually cover an area of the order of a few square km or perhaps less.

Table 4.1.1 shows the main categories of information depicted on engineering-geological maps of different scales (Matula, Pašek, 1966). The classification shown was that recommended in countries of the former Eastern Bloc. The Table is complemented by engineering-geological design and survey stages corresponding to respective scales of representation. The scales appropriate to individual stages of the engineering-geological survey are shown.

Table 4.1.1 Categories of information depicted on EG maps and the corresponding survey and design stages

Map scale	**Rocks**	**Hydrogeological conditions**	**Geodynamic phenomena**	**EG zoning**	**Survey and design stages**
Synoptic map 1:200 000 – 1:100 000	Geological complexes and geological formations depicted	Depth of 1st aquiferous horizon	Area of phenomena indicated by arbitrary symbols	Regions, areas, zones	Inventory survey; multi-purpose utilization of river (preliminary study)
Basic map 1:50 000 – 1:25 000	Individual rock types defined using lithological criteria, mineralogy and texture	Delineation of 1st aquiferous horizon and depth of GWL <2 m, 2–5 m, 5–10 m, >10 m	Delineation of areas affected by certain types of geodynamic phenomena	Area, zones, sub-zones, and/or districts	Orientation survey; project design and economic feasibility
Detailed map 1:10 000 – 1:5 000	Rock types defined using engineering-geological criteria	Hydroisohypses of 1st aquiferous horizon at 1m intervals at maximum water level	Depiction of areas of individual phenomena	Zones, sub-zones, districts	Preliminary and detailed surveys; preliminary and master engineering projects

A geological formation is the fundamental mappable unit of the lithostratigraphic hierarchy. The geology of the pre-Quaternary bedrock is divided into such mappable formations. A formation consists of a set of rocks of similar facies and composition formed during a defined period in the development of that part of the Earth's crust. These characteristics enable one formation to be distinguished from adjacent rocks of another formation so that the geological contact between them can be mapped. In some cases there is no practical advantage in grouping supracrustal rocks into geological formations.

A geological complex is a lithostratigraphic unit composed of a range of different rocks formed and consolidated during a given tectonic cycle; usually it will have a complicated structure. Such complexes often form the geological basement in deeply eroded continental areas. It is useful to distinguish groups of rocks within a complex that are of similar genesis and composition. Individual rock types within geological formations and complexes are distinguished lithologically using mineralogical, textural and structural criteria.

Engineering-geological rock types are a sub-set of lithologies distinguished by the homogeneity of their engineering-geological properties with respect to the construction of the dam and ancillary facilities. In certain cases, the lithological type will be identical with the engineering-geological type. For mapping purposes, territorial units on the map of engineering-geological zoning are defined on the basis of engineering-geological assessment as follows:

- Engineering-geological regions broadly defined by the homogeneity of the geological structure and its geotectonic development, e.g. the region of Carpathian flysch, the region of Neogene volcanic rocks, etc.;
- Engineering-geological areas defined by the homogeneity of geomorphological units of a higher order (macro-relief), e.g. the area of flysch highlands and the area of flysch depressions;
- Engineering-geological zones defined by the uniformity of composition of the subsoil, e.g. the zone of marine sandy sediments, the zone of deluvial sediments, etc. If necessary, we can distinguish sub-zones within zones according to differences in the stratigraphy or in the thickness of individual layers; and
- Engineering-geological districts defined in terms of the uniformity of hydrogeological conditions, or the influence of geodynamic processes, e.g. a district more susceptible to seismic activity.

An example of a map of engineering-geological zoning is that prepared for the Corojo dam (Fig. 4.1.1). The designer proposed three alternative sites for the Corojo dam. Of these, the profile B–B´ was recommended as the most suitable. A geological section of this is shown on the right of the figure. A map of engineering-geological zoning at a scale of 1:10 000 was compiled covering the same area as the detailed engineering-geological map. The whole area of the planned dam, backwater and ancillary structures lies within the anticline of the Sierra Maestra Mountains (I) and is subdivided into two sub-regions, I-A and I-B. The first is an area of mature relief in which erosion and denudation has produced wide valleys on a surrounding peneplain with low hills and well-developed river terraces, and the second is an area of immature relief characterized by elevated ridges and deeply incised river drainage patterns without the development of river terraces. These sub-regions were further subdivided into zones I-A1 – a zone of wide valleys, I-A2 – a zone of low hills, I-B1 – a zone of elevated hills and ridges, and I-B2 – a zone of hills and ridges of medium height. The geological, hydrogeological and geotechnical characteristics of each of the defined zones are described in detail in an accompanying legend. This includes information about the predicted permeability of rocks, the influence of geodynamic processes such as weathering and erosion and slope failures and instabilities on different types of rocks, drawing attention to potentially detrimental effects on the construction of the dam and ancillary structures and on the future reservoir area.

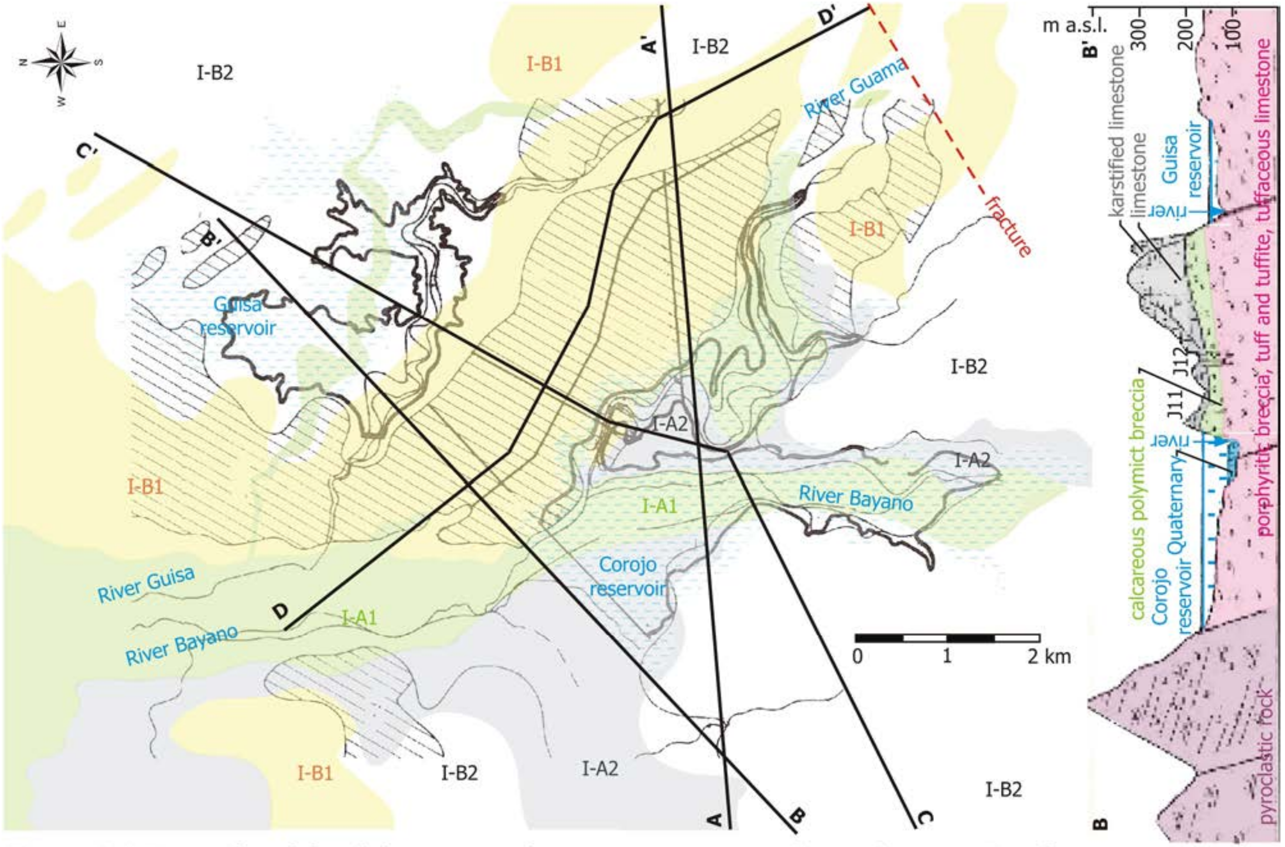


Fig. 4.1.1 Example of the delineation of an engineering-geological region I, sub-regions A and B, and areas A1, A2, B1 and B2 in the vicinity of the Corojo dam, shown in relation to a geological cross section (B-B') through the area (after Horský and Reyes, 1980)

The map showing the engineering-geological zoning of the banks of the Iskar, Kirdzali, Ivaylovgrad and Topolnitsa water reservoirs is another example of this type of classification (Horský, Simeonová, Spanilá, 1984). The banks of each of these reservoirs are subdivided into the following categories: abrasion, abrasion-accumulation, landslide (including toppling), karst, and marsh. The classification of the banks into engineering-geological zones based on their physical coherence (unconsolidated clayey, semi-rock, and solid rock) and their susceptibility to erosion and washout (very rapid, rapid, average, slow, and very slow) proved to be of great practical value in predicting the behaviour of the banks after the reservoirs were filled up (Fig. 4.1.2).

The procedure for recording and compiling engineering-geological observations on a map must take account of two requirements. The first requirement is that the map must be comprehensive. This means that all the relevant observations must be included on the map. The second requirement is that the map must be clearly legible and understandable so that it can serve its main purpose, that is, to guide the design and construction of the dam and the ancillary facilities. In recent years, the process of analysis has been greatly facilitated by the compilation of maps using digital formats that can be managed as layers within a geographic information system (GIS). This has now become the usual procedure on most projects, but the quality of the data and the interpretation still depends on the skill of the engineering geologist who makes the observations and supervises the testing and drilling operations in the field.

By the stage at which the design of a dam starts, an engineering-geological map of the area of interest will already have been compiled. This map will cover the whole service area around the body of the dam, that is, the area in which the plant and services required during construction will be located. The map must also include the future reservoir area where there will be geodynamic effects caused by the water filling the reservoir and the fluctuation of the water level in it, and because the extraction of construction materials from this area must also be taken into account. The construction of a water reservoir usually requires that roads are realigned. At the beginning of the design work the exact plan will not be known, so it is desirable to map the area as far as the watershed of the drainage which will be affected by backwater. By mapping to the limits of the watershed, all the geodynamic processes existing in the area can be recorded and possible changes can be predicted. This procedure also enables rational choices of sites for the extraction of local construction materials.

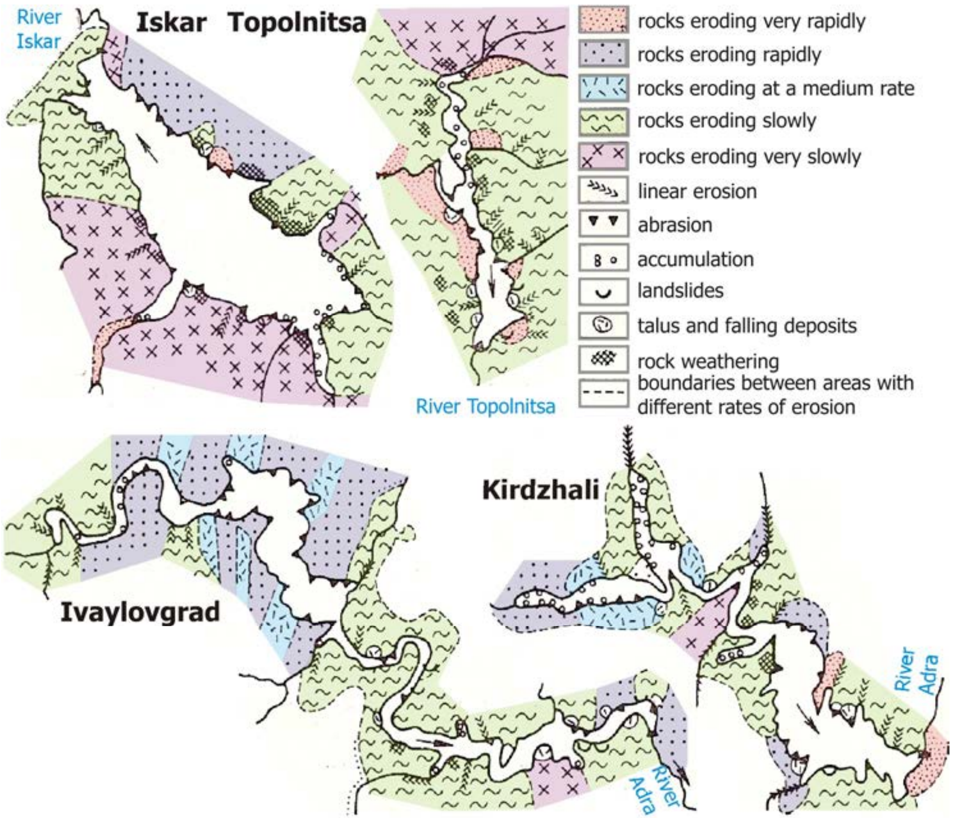


Fig. 4.1.2 Maps showing the engineering-geological zones on the slopes surrounding four water reservoirs in Bulgaria (after Horský, Simeonova and Spanilá, 1984)

An engineering-geological map is the tool used by the engineering geologist to compile the information that will be used to interpret all the conditions in the area chosen for the construction of a dam so that a final report can be submitted to the design team. It also forms the basis for planning the detailed engineering-geological surveys required for the dam itself and the associated ancillary facilities. Because the map and its legend are the references which guide communication between the geologist and the designer, the legend to the engineering-geological map must classify and explain the significance of all the features depicted on it. To increase the informative value of the map, supplementary explanatory notes can be attached. These will tabulate the main characteristics of the rock environment, the geodynamic phenomena, engineering-geological zones, hydrology, etc. Above all, the information on an engineering-geological map of any scale must be accurate and clearly legible so that it can be used with confidence by all members of the survey and design team involved in the project.

4.2 The Compilation and Presentation of an Engineering-Geological Map

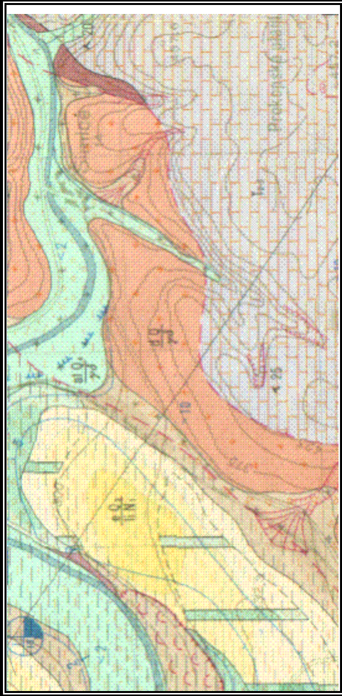
An engineering-geological map can be produced in the traditional way by compiling observations directly on a topographic map, aerial photograph or plan as work proceeds in the field. These observations will also be recorded in the field notebook referenced by the location and the date when they were made. Appropriate notes describing and illustrating the geological and geotechnical relationships at the documented sites will be used to support the measurements made in the field. This procedure is simple and direct. In recent years, new techniques for recording observations on hand-held computers referenced to differential GPS and digital terrain models have evolved. By this means observations can be fed into a central database as field trips are carried out and the observations used to create a GIS. This procedure depends on sophisticated instruments that are expensive. In both cases however, the quality of the input information depends on the professional skills of the geologist responsible for gathering the primary data. The advantage of working with digital information in a GIS is that data can be classified and manipulated rapidly to provide the type of information most relevant to the particular problem under consideration. The different layers of information can be treated separately or in combination and studied on a monitor. Working copies can then be printed rapidly for distribution to drillers, geologists, engineers and technicians working on site. The data can also be updated and incorporated much more rapidly than would be possible by conventional means.

To draw attention to the relationship between the geological features observed in the mapped area, it is desirable to present as much of the relevant data as possible on a single sheet of the engineering-geological map. However, when the map becomes unclear and difficult to read because the data is too crowded, it is appropriate to filter out certain categories of data and depict them on auxiliary map sheets. However, in all cases it is recommended that areas covered by Quaternary drift (alluvium, till, loess, etc.) and the areas in which pre-Quaternary rocks crop out, or are exposed at surface, should be shown on the same map sheet.

On the map of engineering-geological conditions, only the observations that relate directly to the construction of the dam and peripheral facilities or that add information about the geological structure of an area should be depicted. As far as possible, the geological formations that are identified will be classified according to their age, origin and composition and according to their metamorphic and tectonic state. In addition to showing the areas covered by Quaternary drift, the map should also show the depth of cover and the thickness of the separate units of the sedimentary cover where more than one unit is present. In situations where a number of different formations make up a composite lithostratigraphic sequence, it may be practical to combine a number of rocks with similar physical and mechanical properties into a single mappable unit for engineering-geological purposes. Geological contacts cropping out or exposed at surface will be depicted by a solid line, and geological contacts submerged under the terrain will be depicted by a dashed line.

Areas where formations of the pre-Quaternary basement crop out at surface are indicated using appropriate colours and symbols that show their stratigraphic age and origin. Igneous and volcanic rocks will be indicated according to their composition and age, if known, using conventional colours and symbols. The areas of the bedrock below the Quaternary cover are indicated by dark-grey hatching and appropriate symbols. The preferred convention for the use of colour and symbols will be that recommended by the International Commission on Stratigraphy (ICS) of the International Union of Geological Sciences (IUGS).

The depth of the pre-Quaternary basement below the overlying cover is depicted by hatched lines of different thicknesses. Depths up to 5 m and from 5 to 10 m, or greater than 10 m below surface are usually indicated. The boundary between the areas of hatching of different thickness is not drawn in. If rocks of the pre-Quaternary basement lie at depths greater than 10 metres below surface over most of the area of the mapped sheet, then it is recommended to use this way of representation by a hatch for the third (from the surface) complex of rocks of cover formations. The depth and identity of the rocks forming the pre-Quaternary bedrock can be shown by dashed contours of an appropriate colour on the same map sheet or on an auxiliary map sheet.



group	Proces of Formation	Geological Age	Supeficial Deposits	Stratigraphic index	Determination of thickness or depth of rock						Thickness [m]	Characteristics of soil or rock
					Thickness in first genetic complex [m]			Depth of surface in second genetic complex [m]				
					< 2	2 to 5	5 to 10	< 2	2 to 5	5 to 10		
Surface	Quaternary	Holocene	Fluvial	dlQ_4							2–4	sandy clays of grey colour
											1–2	well-sorted gravels
		Pleisto-cene		dlQ_3							6–8	clayey-sandy gravels of higher terraces
			Aeolian	eQ_3							10–12	porous loess
			Deluvial	dQ_{2-4}							2–4	clayey slope loams of brown colour
											3–5	slope debris with stone blocks
bedrock	Tertiary	Neogene	Lacustrine	IIN_1							50	grey claystones with intercalations of clays
	Mesozoic	Triassic	Marine	T_{2-3}							400	limestones and dolomitic limestones of grey colour
	Palaeozoic		Intrusive	$\gamma\delta$								medium-grained diorites of grey colour

Fig. 4.2.1 An example of an engineering-geological map

Areas of rocks of the uppermost cover formation are indicated by a plain colour or by a coloured ornament. This colour identifies the lithology and genesis of the formation as indicated on the legend of the map. So that the map can be read easily, following the convention for formations of Quaternary cover, pale colours are used. The thickness of the cover formations is usually indicated by using three shades of the same colour that correspond to thicknesses of less than 2 m, 2 to 5 m, and 5 to 10 m respectively. The more intense the colour used, the greater the thickness. In places where the thickness is greater than 10 m, the most intense colour is still used, but the hatching of the pre-Quaternary bedrock beneath it is omitted. The lithological character and genesis of the second formation of cover below surface are depicted in a similar way but by using vertical striping. In doing this, the ranges of thickness that are mapped can be modified to suit the local geological conditions. An example of the way in which such a map is constructed is shown in Figure 4.2.1. This map was compiled at a scale of 1:25 000 and is appropriate for the first stages of a survey for a dam and its ancillary structures.

The description given above applies to the traditional procedure for the compilation of information on paper maps, as well as to the compilation of a Geographic Information System using hand-held devices, however the manipulation and editing of data within the digital format of the GIS is a much more rapid process and there is also the advantage that different layers of information can be combined to suit specific needs or stripped away to simplify the interpretation and editing of particular types of information.

In areas where tectonic deformation has taken place, it will be important to compile a map of the structural features affecting the site where the construction of a dam is planned. Where the structure is complicated and numerous structural observations are made, a separate structural map or layer of the GIS will be compiled. If the structure is not too complicated, and the number of tectonic observations is limited, these can be depicted together with the other geological information on the basic sheet of the engineering-geological map.

The main categories of structural measurements depicted on the map will be the strikes of bedding, cleavage and foliation, the main systems of joints and fractures, and faults with their amounts and direction of throw, the direction and plunge of fold axes and the dip of their axial planes, and the orientations of crenulations and lineations. All these will be depicted using appropriate symbols that are explained in the legend of the map. The angle of dip on planar features and the plunge of linear features are marked by a number beside the symbol. The direction and amount of throw on faults is also indicated, as well as the sense of displacement and the width and type of gouge. If the exposure is good and numerous measurements can be made within a limited area, the direction and angle of inclination of a particular structural feature shown at a particular location on a tectonic map can be the statistical average of a number of readings. By using stereograms and rose diagrams, the structure in different sectors of the area can be analyzed. Special attention is given to the risks posed by recently active faults or structures that could be reactivated as a result of the planned construction work.

When the hydrogeological conditions in an area have a fundamental bearing on the design and construction of the dam, a significant amount of information about the hydrogeology of the area must be gathered. In this case a separate map or GIS layer depicting the hydrogeology of the area will be compiled. If the hydrogeological conditions are not complicated, or if only limited data is available, the hydrogeological data will be depicted on the engineering-geological base map. The convention is to mark hydrogeological information on the map in blue. Besides showing watercourses, springs and waterlogged places, the depths of the groundwater table are indicated by using hydroisobaths marking the ranges of less than 2 metres, 2 to 5 metres, 5 to 10 m, and greater than 10 m depth. Symbols are used to mark the type of chemical aggressivity of the groundwater. If a significant number of laboratory and field tests of permeability and pumping tests have been made, then areas of the rock mass having the same permeability or other hydrogeological properties governing seepage from the planned reservoir will be depicted. Important geodynamic phenomena are marked in red on the engineering-geological map using appropriate symbols and ornaments that are explained in the legend. Areas of landslides, debris flows, rock falls and other instabilities will have been identified by carrying out a study of satellite images, aerial photographs, and topographic base maps and by observation in the field.

The characteristic engineering-geological properties of the rocks composing each of the geological formations and complexes that have been mapped are systematically arranged in tables and in explanatory notes. The map of engineering-geological conditions must be accompanied by one to two representative geological sections along profiles that demonstrate all the main engineering-geological factors in relation to the structure and geological composition of the area. Usually, one geological section will lie along a profile through the selected dam site or one of the alternative sites.

The map of engineering-geological zoning depicts those areas of rocks with the same history of geological development that also share similar engineering-geological conditions. For example, a river basin can be zoned in relation to the factors governing the construction of a dam, and the backwater area of a reservoir can be classified in terms of potential seepage; the susceptibility of the area to geodynamic processes after the reservoir is filled; the potential for the banks of the future reservoir to re-shape or degrade; the susceptibility of the surrounding slopes to sliding, seismicity, etc. The areas covered by units with similar engineering-geological characteristics (zones, sub-zones, districts) are depicted by a specific colour or a hatched ornament corresponding to particular engineering criteria, paying particular attention to dangerous phenomena (in red). The convention is that unsuitable or dangerous areas are marked in red. If an engineering-geological map is processed in a GIS, the turning on and off of selected layers of information enables the factors governing the engineering-geological conditions in a given zone to be analyzed systematically.

Other data relevant to an assessment of the suitability of an area for a hydraulic construction will also be marked on the map using appropriate symbols and ornaments. In particular, resources of construction materials and for mining operations, mineral deposits and their protection zones, protection zones in general, areas of increased seismicity, sites of special interest, etc., will be depicted. The map must therefore include a full legend in which the geological identity, geomorphological features, engineering-geological properties and hydrogeological characteristics of the formations mapped in the area are summarized, together with an overview of the resources of construction materials suitable for use in constructing the dam and its ancillary facilities.

All the observations used to compile an engineering-geological map must be properly recorded. The natural or man-made sites at which observations and measurements are made are the Documented Points (DP). The coordinates of these sites must be permanently recorded and each point must be given a unique number so that the data can be spatially located on the topography of the surveyed area. The points will be marked directly on field slips together with the relevant measurements and also recorded digitally using a GPS or by writing in a notebook. A distinction is made between points that have been accurately surveyed and points for which the position is known approximately (e.g., older, unsurveyed boreholes). Symbols linked to the documented points indicate what types of measurements, field or laboratory tests, etc., have been made at that point or on material collected from it. The map also incorporates information about the occurrence of mineral raw materials, especially natural construction materials, and places where they have been, or might be mined. The areas designated for further investigation of inferred resources of construction materials suitable for the dam, and the areas for which reserves of construction materials have already been calculated will also be shown.

The frequency of documented points on the engineering-geological map depends on the level of detail of the survey and that will be determined by the complexity of the geology in the area of the dam project. Other factors that affect the number of documented points required are the siting and technical complexity of individual buildings, the location and extent of prospective deposits of construction materials, etc., but the number should at least match the scope set for the engineering-geological survey at the scale of the map that is chosen. The geological institutes and regulatory authorities of certain states publish guidelines prescribing the number of natural or man-made points per square kilometre that should be documented, depending on the scale of the map, the complexity of geological conditions, and the accessibility of the terrain. These recommended frequencies of documented points are given below in Table 4.2.1.

Table 4.2.1 Number of DP on an EG map

Map scale	Approximate number of documented points per km^2	
	Engineering-geological conditions	
	Simple	Complicated
1:200 000	1	1-2
1:50 000	4	6
1:25 000	8	14
1:10 000	15	25
1:5 000	25	50

Of the total recommended number, at least a half of the documented points will refer to rocks of the pre-Quaternary basement if they lie at a depth less than 10–15 metres below the surface. In mapping the immediate area of a dam or the main ancillary structures it is usually necessary to increase the frequency of documented points.

The principles and the details of the methods recommended for the construction of engineering-geological maps are described in “Guideline No. 1” on engineering-geological mapping, published in the former Czechoslovakia in 1989. This explains the methods used to compile synoptic engineering-geological maps at 1:100 000 and smaller scales, as well as for maps at scales of 1:50 000 and 1:25 000, and for detailed maps at 1:10 000 and larger scales, and for special maps. The conventions for depicting pre-Quaternary rocks and Quaternary cover, hydrogeological conditions, geodynamic features, tectonic conditions and seismicity are all described. The procedure for distinguishing engineering-geological zones on maps at different scales is also described. The Guideline recommends that the basic rules are observed, but some modification is permitted in compiling multi-purpose maps. Specific changes in the ornaments and symbols used are permissible if the quality and legibility of the final map are improved.

The principles and recommended procedures for the compilation of an engineering-geological map described above apply equally to classical engineering-geological maps and the digital compilation of maps using GIS. The use of GIS enables output to be tailored specifically to the needs of the engineering geologist or to the engineer and designer. The software and hardware in use is evolving rapidly and the conventional demands of geologists can easily be met, but the scope for innovative applications is immense. One of the greatest advantages is the speed with which maps can be edited and presented on the screen of a computer, or printed for use and distribution in the field and in the office. Similarly, more or less immediate access can be gained to topographic maps and information held in databases covering the area of a project. Digital methods have certainly enabled more efficient storage and manipulation of data, but ultimately the success and safety of a project depend on the quality of the data and that ultimately depends on consistent and careful measurement and observation made in the field by the engineering geologist responsible.

4.3 Work Flow for Compilation of Engineering-Geological Maps

The fundamental principles on which engineering-geological mapping of dam sites is based are the same as those used in carrying out engineering-geological mapping for other purposes. Also, the types of information depicted on engineering-geological maps at different scales are essentially the same, but the frequency and detail of the observations increase progressively as the scale of the map decreases.

Nevertheless, because of the specific requirements for information determined by the different stages of development of a dam project, the work flow should follow a definite plan (Bůžková, *et al.*, 1964).

The compilation of engineering-geological maps can be divided into four basic stages: preparatory work, fieldwork, laboratory work, and processing of results and compilation of an engineering-geological map at the appropriate scale.

4.3.1 Preparatory Work

Preparatory work is undertaken prior to fieldwork on the selected site. The geologist contracted to make the engineering-geological map must first carry out a thorough evaluation of all available information relating to the geology, geomorphology and hydrogeology of the area of interest. The results of previous geological mapping are re-drawn on a topographic base of suitable scale, though not necessarily the same as that on which the new engineering-geological survey will be compiled.

The important types of information obtained from archival sources that will be transferred to the new engineering-geological base map include the details of previous underground workings and subsurface exploration (e.g., drilling). In addition, the locations of documented points from older surveys will also be marked, together with previously identified groundwater springs, and exposures created by extraction of minerals or aggregate (quarries, borrow pits, sand pits, etc.). An integral part of this process of compilation is the creation of a database including all relevant information contained in archived technical reports and earlier publications. All available information on the geological formations and the physical and chemical properties of the rocks and waters in the area of interest are tabulated systematically.

When the review of previous work is near completion, field trips will be made by the geologist in order to verify the features described in the earlier work. This reconnaissance also enables the geologist to become familiar with the topography and exposure across the area to be mapped. At this stage the basic criteria used to distinguish different rock types and formations will be established by reference to previous studies and by direct observation in the field. Account will be taken of published information about the geological age and the stratigraphic subdivision of the rocks in the area. Where the stratigraphic status and lithology of formations has been established as a result of detailed studies by experts of a national geological survey, it will be appropriate to use the existing classification, as far as possible.

If doubts are raised about the identity of certain rock types, it will be necessary to consult with a specialist and to carry out petrographic studies that will provide an unambiguous answer for the purposes of the planned engineering-geological survey. In areas where the geology is particularly complicated, it is a useful procedure to make a representative collection of rocks from the area for reference purposes. For example, when carrying out the survey for the Centro Cuba PSHEP, the teams of geologists, with the exception of the chief manager, were changed several times. The reference collection of rock types enabled the new team to begin with a clearly established understanding of the different rocks in the project area and to use the same criteria for identifying and naming them in the field. This enabled the new team to proceed smoothly with the survey and to construct a number of geological cross-sections that were verified by a preliminary geophysical survey.

Preparatory work ends with the submission of the plan for systematic engineering-geological mapping that will then be carried out. The amount of survey work to be undertaken in the field will be estimated, together with the preliminary numbers of field and laboratory tests of soils, rocks and water, etc., and the profiles designated for geophysical surveys. The final scope of the work will be determined as field-work progresses, depending on the feedback obtained as the results of observations and test measurements are progressively compiled.

The plan of work must define the extent of the area to be mapped, bearing in mind that the limits will extend well beyond the footprint of the backwater area of the reservoir. This is because the engineering-geological map must take account of the impact that construction work will have not only on the stability of the natural environment. For instance, large fossil slope failures can be reactivated and new instabilities created because of changes in the groundwater regime, saturation or suffosion, etc. Also it is sometimes necessary to increase the mapped area to include the outcrops of rocks near the construction site that could be used for construction material or to provide additional information on which plans for recreational developments in the vicinity of the water reservoir can be based. In certain types of terrain, notably in karst regions and in regions where there are significant thicknesses of permeable rocks, the scope of the map must also be extended to include the area affected by seepage of water from the reservoir into the surrounding formations, and to include neighbouring streams if there is a likelihood of hydraulic communication between them.

In the plan for mapping work, the scale of the map will be defined. This is mostly determined by the scale of the most recent available topographic base map and by the complexity of the geology of the area. A scale of 1:5 000 is usually chosen, but a scale of 1:10 000 is appropriate in areas with simple geology. As a rule, these scales are suitable for compiling maps on which to base the design of the project and to complete a preliminary plan. Engineering-geological maps for more detailed purposes are compiled on topographic base maps of larger scale, but usually cover only selected parts of the area of interest. For example, the area covered by the profile of a dam would be mapped at a scale of 1:2 000 or 1:1 000. In the preparatory stage of work, as well as during field mapping, full use will be made of aerial and satellite images that enable the geomorphological features and areas of intense geodynamic activity to be mapped precisely. These will include ancient slope failures and active landslides, rock falls, debris flows, water saturated areas and actively eroding gullies, etc.

The importance of preparatory work should never be underestimated. Inadequate preparation usually leads to inefficiency caused by lack of awareness of the scope of previous work so that unnecessary repetition and difficulties of interpretation could occur during mapping. Under the pressures imposed by the schedule of work, the geologist may then be reduced to making compromises that would not have been necessary if a careful preliminary assessment of the existing information and a full analysis of the problem had been made beforehand. Notwithstanding this important caveat, commercial pressures can place an engineering geologist in a position where the time and resources to make a full preliminary assessment are simply not available. For example, when the Czech team arrived on site to undertake the engineering-geological survey for the Centro Cuba PSHEP, it was discovered that there was no basic geological map of the area. In such circumstances the success of the project depended on an immediate appraisal of the situation so that a strategic plan could be improvised using all the geological skills and experience that the team could offer.

4.3.2 Fieldwork

Fieldwork can be divided into three main categories:

- Surface surveys, i.e. the direct observation at surface of diverse geological, morphological and hydrogeological conditions that are recorded and mapped;
- Subsurface surveys, which include, for example, mapping boreholes, testing pits, trenches and adits, etc.; and
- Special surveys, for example, using geophysical methods, hydrogeological tests and field testing of rocks, etc.

The basic data used in the construction of an engineering-geological map is collected by making observations and measurements at natural geological exposures on the topographic surface. These are recorded and numbered systematically as documented points with their co-ordinates. Such natural exposures include those formed on outcrops of pre-Quaternary basement rocks, natural sections in rocks of the Quaternary cover, including the upper layer of the Quaternary cover that forms the top surface of the area under investigation, and also springs and wells. Man-made exposures formed by drilling, trenching, and underground exploration must also be checked and recorded during fieldwork. These will include, in particular, all older hydrogeological boreholes and drains and their outlets. The normal procedure would be to review and record the reported location of these during the preparatory stage of work, but the sites will be visited in the field to confirm the reported locations.

At each geological exposure photographs and sketches will be made, together with a systematic record of the petrographic composition and texture of the rocks found there. The orientation of structures such as bedding, cleavage, foliation, lineations, folds and joints are measured. Mineralization and alteration, if present, are also described and the state of weathering of the rocks is noted. The strength of rocks is also estimated. The physical-mechanical characteristics (water content, consistency, bulk density, grain size, etc.) of the rocks of the cover formations are also recorded. The descriptions will be made in an agreed format and using consistent terminology that can be entered into the database of the project and depicted on the appropriate layer of the engineering-geological map or GIS.

In the case of natural springs, location, altitude and the type of spring and its yield are also recorded, as well as the extent and character of the spring area. Using information gathered from local people, the fluctuation in the yield of springs is estimated and the current use and future plans for use of the water are recorded. In the case of springs that form an important contribution to local water supplies, the location of the spring will be geodetically surveyed and, if necessary, water samples will be collected for laboratory analysis. Records are made of the total depth of all wells together with the depth below surface to the water level in each well and the purpose for which the water is used. The range of fluctuation in water level reported by the user is also recorded. The co-ordinates and elevation of the collar are also determined, if necessary by making a geodetic survey. Samples of the water from each well will also be collected for analysis in the laboratory. Disused boreholes, wells, drains and their outlets are surveyed in the same way.

Increasing reliance is now being placed on the use of georeferenced aerial photographs and high-resolution satellite imagery to enable surface exposures, boreholes, wells and other man-made excavations to be located precisely on maps and plans. Exact co-ordinates are required so that the information gathered during the engineering-geological survey can be entered in the project database and manipulated in digital formats so that it can be displayed graphically within a GIS (Fig. 4.3.1).

The analysis of geomorphological features and the identification of related geodynamic processes are crucially important in an engineering-geological survey. The detailed study of geomorphology is the key to assessing the stability of an area. This will govern the choice of suitable sites for the dam, and enable the stability of the banks of the planned water reservoirs to be predicted and safe routes for service roads and water canals to be chosen. A study of the geomorphology of a dam site must take account of relief in the rocks of the pre-Quaternary basement concealed beneath the Quaternary cover as well as that exposed on the recent surface. The evolution of individual geomorphological features must be related to the processes responsible for their formation. For example, the stages of development in the erosion of valleys and the accumulation of alluvial deposits within them exert a fundamental control on the hydrogeology and the changes that are likely to be caused as the work of construction proceeds (saturation of the banks of water reservoirs and canals, water leakage from reservoirs and canals, etc.).

Well 268 (non-functional)

N: 45.76293° (45° 45′ 46.6")
E: 106.28040° (106° 16′ 49.5")
Z: 1,423 m a.s.l. - GPS bar
Final depth: 70 m

0.0 - 21.0	Detrital material with loams, 15.0-21.0 with sand
21.0 - 25.0	Grey, compact clay
25.0 - 65.0	Detrital material with loamy-sandy fill, 37.0-43.0 and 52.0-64.0 with sand
65.0 - 70.0	Grey, compact clay

q	l/s.m	1.1
CM	g/l	0.73
Screens		
15-21 + 37-43 + 52-64: Ø 219		

Fig. 4.3.1
View of an abandoned well with the log describing the sedimentary sequence and physical parameters (a photo by P. Bláha - 2004)

An integrated geomorphological study of any area should include the chronology of development of the relief in relation to the geodynamic processes responsible. The patterns of occurrence and the relative ages of the various processes must be studied in detail so that predictions can be made about the likelihood that non-active or fossil processes will be triggered into new activity by the work of construction. The chief geodynamic features to be recorded on the engineering-geological map are as follows:

- Slope failures and areas susceptible to failure;
- Areas of karst phenomena;
- Erosional features, ravines and proluvial cones below slopes affected by erosion;
- Areas affected by subsurface erosion (suffosion);
- Areas of loess where collapsibility due to saturation has occurred;
- Saturated areas; and
- Deeply disturbed slopes where the rocks have been deteriorated so that the mechanical behaviour of the subsoil is changed, etc.

Based on the results of such a study, it is possible to predict the progress of development of relief and the type of geodynamic processes, especially slope failures, suffosion, etc., that might be initiated or reactivated as construction work proceeds. In areas affected by Pleistocene glaciation and periglacial phenomena, attention must be given to the distribution of glaciofluvial, glaciolacustrine and glacial sediments on slopes, especially the products of periglacial solifluction that can be dangerously unstable if the hydrological regime changes.

In mountain areas and in areas with a complicated geological structure, a geomorphological survey must take account of the results of the latest advances in geophysics and rock mechanics. Certain processes which govern the erosion of mountain valleys can be understood in terms of unloading and stress release in the rock mass. The key systems of fractures can be traced on aerial photographs and satellite images and their effects on the mechanical behaviour and stability of the rock mass can be analyzed.

During mapping, the geologists responsible must remain constantly aware of the factors that are likely to affect the technical procedures employed in the construction of the dam and its ancillary facilities. Account must be taken of problems that have occurred on other projects in similar types of terrain. This will constrain the methods employed for foundation engineering and draw attention to any factors that could lead to structural failures. The triggering of geodynamic processes by the construction work, and the quarrying of local resources of construction materials, etc., must all be considered carefully. Experience of these types of problems obtained from previous projects can be used to advantage when assessing the engineering-geological conditions in a new area of interest.

In all cases, the geologists responsible for mapping should be continuously aware of the inter-relationship between the separate features observed and the various measurements that are made, in particular:

- The lithological characteristics, bedding and the mechanical state of the individual engineering-geological rock types identified, and their structural and stratigraphic position within the sequence of geological formations or more complex lithostratigraphic units that crop out in the area of interest;
- Tectonic processes that have affected the rocks in the area of interest, in particular the mechanical changes produced by folding, fracturing, cleavage, foliation and faulting. The systematic analysis of these patterns will be used to interpret the geological structure of the area and the sequence of tectonic events responsible;
- Geomorphological features of the area, bearing in mind the relationship of the present relief to the types of rock and the tectonic structures that affect them. The chronology of evolution of the landforms must be interpreted in relation to the geodynamic processes responsible and the dormant and active processes must be identified so that the likely impact of the planned construction work on the geomorphology and hydrogeology of the area can be predicted;
- Hydrogeological conditions and changes likely to be caused by construction work are of particular importance so the following categories of information must be gathered systematically:
  - Classification of all types of groundwater in the area (using the relevant standards);
  - Conditions of groundwater infiltration, circulation and drainage, inter-relationship of individual types and horizons;
  - Groundwater levels and their fluctuation;

  - Chemical aggressivity of groundwater;
  - Current exploitation of groundwater in the area;
  - Existing and potential sources of pollution;
  - Effect of groundwater on karst development;
  - Effect of groundwater on changes in the physical-mechanical properties of rocks; and
  - Requirements for additional special survey and the method to be used;
- Suitability of the rocks in the mapped area for use as construction material; the types of rocks accessible are classified in terms of their properties based on previous experience of use of similar rocks as construction materials. Subject to a satisfactory preliminary evaluation, further survey work, field tests (e.g., compaction tests, etc.) and sampling for laboratory tests of physical-mechanical properties will be undertaken; and
- The existence of other exploitable mineral raw materials in the mapped area. The geologist has a duty to notify relevant organizations of such occurrences and, at the same time, to think about the potential sterilization of such deposits by the planned dam and its ancillary facilities. Problems of this type can obviously give rise to 'conflicts of interest', though in most circumstances the regional or national planning authorities would already have taken a strategic decision about such matters.

Based on an evaluation of the results of surface mapping, the engineering geologist will submit proposals for a programme of subsurface exploration work. The location and scope of this work will be chosen to provide the additional information required for a confident interpretation of the subsurface geology of the area, in particular along the profile of the proposed dam and wherever excavations for foundations, canals and roads will be made. It will be desirable to carry out geophysical surveys along specific profiles, and to make field tests on rocks, etc. All the geotechnical and geophysical information obtained in this way will be used to improve understanding of the physical-mechanical properties of the rocks, the tectonic structure of the area and the depth of weathering and mechanical disintegration of the pre-Quaternary basement, as well as the thickness of cover formations.

Proposals will be made for the monitoring of active geodynamic processes so that their effects and the rates of movement can be determined (the speed of advance of landslides, initial measurements of the abrasion and erosion of the banks of water reservoirs and canals, etc.). Based on the study of hydrogeological conditions, further special field tests (pumping and infiltration tests) and observations of the groundwater regime will be made. The water levels and the yield of springs suitable for captation will be measured and those that could be affected by water seepage from the reservoir or canals are identified. These measurements provide qualitative indications of the effects of seepage after the work of construction is completed.

It is often advantageous to use geophysical methods during the early stages of engineering-geological mapping because the deployment of instruments can be rapid and cost-effective. However, the choice of methods must be made bearing in mind the requirement to define sub-horizontal boundaries accurately without using a closely spaced network of measurements. Recently developed methods, such as ground-penetrating radar, do not provide an answer to this problem. Early in the 1970s, when engineering-geological maps at a scale of 1:25 000 were being systematically compiled in the former Czechoslovakia, great attention was paid to this issue. It was discovered that the most effective solution was to combine vertical electrical sounding with symmetrical resistivity profiling. An example of the use of these geoelectrical methods is shown in Figure 4.3.2. The geophysical measurements made on this profile showed that the geological structure was more complicated than anticipated. Based on the change in slope morphology, a change in the lithology of the bedrock had been predicted. However, it was surprising that a high-resistivity layer was detected across the prevailing part of the profile. The first interpretation was that this layer consisted of gravel, but the presence of a horizontal layer of sandstone was not excluded. A borehole was subsequently drilled 30 metres west of the profile. This showed that the first interpretation had been correct.

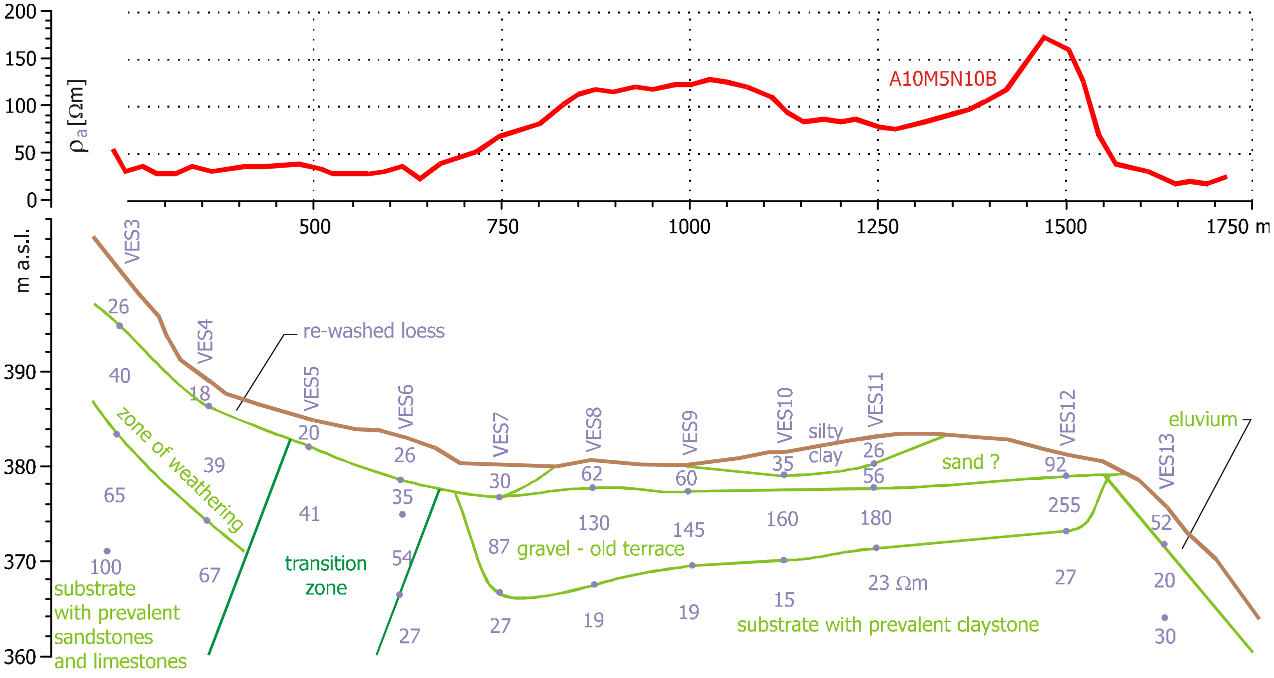


Fig. 4.3.2 Cross section and geophysical profile showing how measurements of resistivity were used to interpret the engineering geology of the site at Frýdek-Místek

The use of geophysical methods for engineering-geological surveys at a scale of 1:25 000 has enabled interpretations of the subsurface structure and geological composition of dam sites to be made with much greater confidence.

4.3.3 Laboratory Work

Laboratory investigations are carried out concurrently with mapping and subsurface exploration work in the field. A range of tests, particularly measurements of certain key properties of rocks such as natural moisture content, consistency, etc., can be made directly in the field (in a field laboratory). More sophisticated testing of rocks is carried out in geotechnical laboratories.

A special part of the laboratory procedure involves testing of solid rocks. This is carried out in conjunction with field tests of rocks made on site (determination of values of static modulus of deformation, shear strengths, dynamic modulus of elasticity determined by geophysical methods, etc.). The programme of laboratory testing is designed to complement the information obtained by field tests. These are primarily tests of cube strength, shear strength, tensile strength, etc. In the case of those rocks which might be suitable for building stone or crushed aggregate, all the tests prescribed by the applicable standard will be carried out. Tests on other construction materials such as sand and gravel will also be made.

In certain cases it is appropriate to carry out petrographic and mineralogical analysis of individual types of rocks and soils. The analysis of clayey soils and rocks is carried out using several techniques that include differential thermal analysis (DTA), X-ray diffraction (XRD), and electron microscopy (SEM and TEM). By these methods the quantitative proportions of individual clay minerals can be determined.

The physical and chemical properties of rocks can be changed by action of weathering and hydrothermal alteration. In some rocks these changes have a deep impact on their mechanical behaviour and permeability. It is necessary to track these changes by drilling holes that penetrate the weathered zone and reach the fresh rocks beneath. By comparing the vulnerability of different types of rocks to weathering and alteration, the extent to which the outcrops of different rock types will be altered can be predicted. Macroscopically, rocks can be classified as strongly weathered, weathered, slightly weathered or fresh. When weathering and alteration is guided by fractures, the extent of this weathering may be deep or shallow. The width of the zone of alteration along the fractures should also be noted. Pervasive weathering and alteration along fractures indicates that groundwater has circulated through the rocks. For purposes of hydraulic engineering, the presence of altered fractures indicates that the major rock permeability is through fissures and cracks.

The survey of hydrogeological features involves the collection of water samples for analysis in the laboratory. A partial or complete chemical analysis of these samples will be made so that they can be classified in terms of their chemical aggressivity. Water which is intended for use as drinking water is subjected to bacteriological screening, in addition to chemical analysis. Certain basic chemical tests on water (temperature, pH, etc.) are usually made directly in the field. Sometimes, chemical analyses are also used to determine the connections between groundwater in different aquifers and surface waters emerging from springs, etc. The results are usually reflected in a hydrogeological map or in derived map layers (maps of seepage, etc.).

4.4 Engineering-Geological Mapping of Special Areas

When carrying out engineering-geological mapping, it is necessary to pay particular attention to special areas where engineering-geological conditions could have an essential effect on the structure of a dam or even prevent its construction. Based on experience, difficulties do arise in areas where the geomorphology and structure are complicated. It is difficult to compile a comprehensive list of all the problems that can arise, but the chief types of complication are listed below:

a) Deep mountain valleys with unstable slopes, where the excavations may destabilize the slopes (e.g., Dalešice, Toktogul, and Guisa dams).
b) Areas which have been affected by repeated episodes of tectonic deformation. The rocks may be cut by several systems of fractures (including tectonic breccia or mylonitic zone) which weaken the bedrocks, reducing their tensile and compressive strengths (e.g., at the Dalešice dam site, tectonic fractures of 1st and 2nd order are accompanied by 11 systems of joints; the Nýdek dam profile – Fig. 4.4.1).
c) Areas of frequent seismic activity, where, if possible, dams should be sited on a stable block of strong homogeneous rock that is not cut by a significant tectonic structure.

d) Karst areas with a deep base level of erosion, where strong development of karst phenomena could be anticipated on the slopes and in the floor of a valley. In such areas patterns of infiltration of groundwater are very difficult to predict and control (e.g., the Corojo dam – Fig. 2.3.26, the Charco Redondo dam – Fig. 2.3.27, and the Foix dam – Fig. 4.4.2).
e) Deeply buried valleys filled by highly permeable accumulations of gravel. Significant amounts of water can be lost by filtration through these permeable sediments. This problem can be difficult and expensive to solve (e.g., the right-bank keying of the Chambas dam profile, the Selenga dam, the El Bosque dam – Fig. 2.3.12, the Slezská Harta dam – Fig. 3.5.2).
f) Areas of former glaciation, where river valleys cut through thick sequences of glaciofluvial or glaciolacustrine sediments which, in addition to permeable beds of gravel, contain lenticular bodies of soft clayey sediments that have the consistency of slurry or fine-grained quicksand. It would not be practical or economical to construct a concrete dam on such a site because the depth to bedrock suitable for a foundation is too great. Construction of dams of any type in areas of former glaciation is problematic (e.g., the Lučina dam; Novosad, Horský, 1973, Fig. 4.4.3).
g) Considerable difficulties may be caused by large slope failures at dam sites (e.g., the slope failure by the Tablachaca dam; Novosad, 1979) or slope failures in the backwater area of a reservoir (e.g., the well-known disastrous Vajont landslide – Figs. 2.3.31 and 2.3.32).
h) Special attention must be paid to the backwater areas of reservoirs where soils or rocks are susceptible to suffosion and bank erosion. After the reservoir is filled, both these processes can lead to big changes in the shoreline and large slope failures along the banks may be triggered (the Orava and Nechranice dams, and the Charvak dam – Fig. 4.4.4). Such changes, together with the silting of the reservoir, may significantly reduce the volume of impounded water, and shorten the life of the water-retaining structure. And

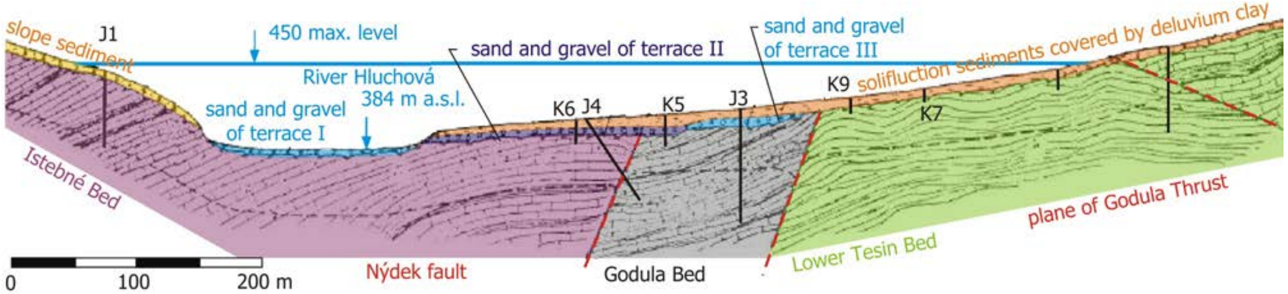


Fig. 4.4.1 Geological cross section along the profile of the Nýdek dam illustrating the complexity of the geological structure

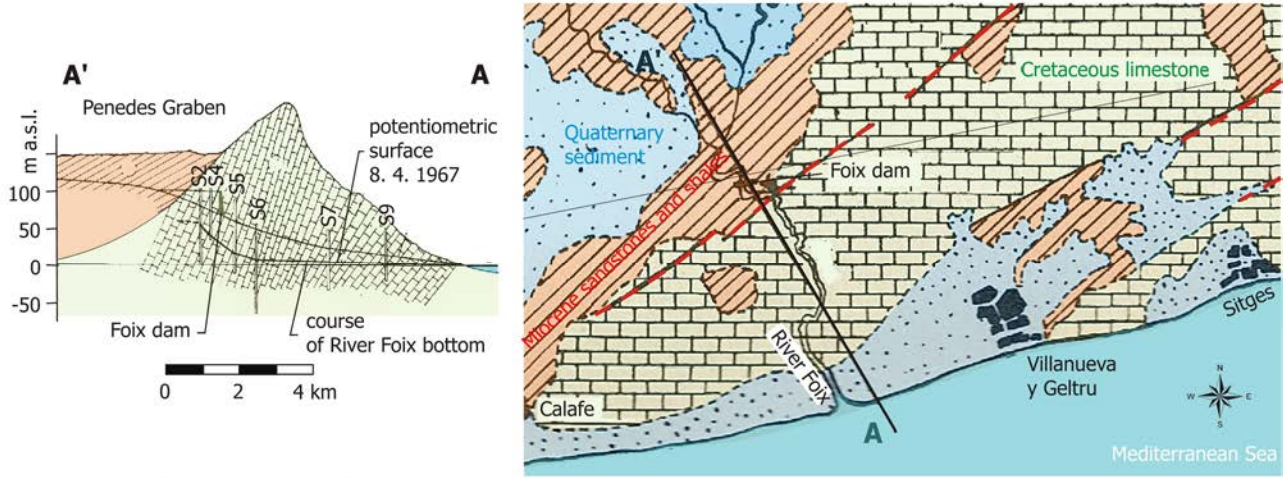


Fig. 4.4.2 Schematic geological map and cross section along the drainage of the Foix dam. The seepage accounted for 86% of the total volume of water escaping from the reservoir (adapted from Madurga, 1957)

i) A particularly careful approach is required when carrying out an engineering-geological survey in undermined areas and areas affected by neotectonic movements.

The Centro Cuba pumped storage hydroelectric plant (Fig. 4.4.5) is an example of a project where the complexity of the local geology had an important influence on the construction of the water-retaining structures. The PSHEP consists of several structures. There is a lower reservoir, occupying the valley on the main course of the River Caracusey, and an upper reservoir which was designed to occupy a natural depression in the slope of the valley that was enlarged by excavation of material to increase the total volume of the water retained. The head reaches 335 metres. The height of the dam on the lower reservoir is designed to be 50 m, and that on theupper reservoir is 58 m. The final plan includes the construction of a surface powerhouse fed from surface penstocks.

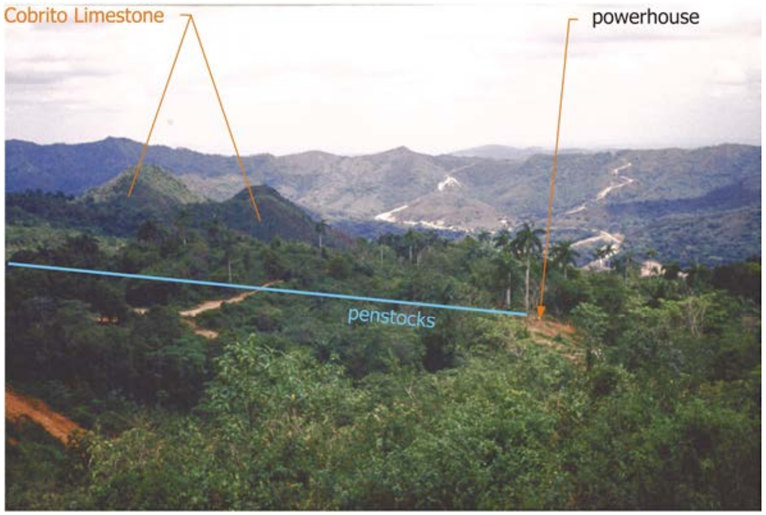


Fig. 4.4.5 General view across the area of the Centro Cuba PSHEP showing the location of the planned penstock and power house (a photo by O. Horský - 1985)

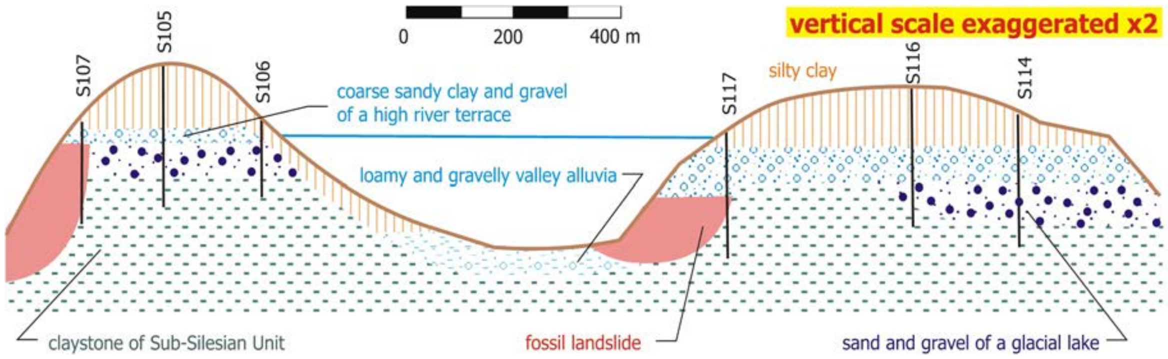


Figure 4.4.3 Geological cross section along the profile of the Lučina dam showing the unfavourable conditions of slope stability and permeability (after Novosad and Horský, 1973)

Figure 4.4.4 View of a slope failure at Mingchukur on the banks of the Charvak dam (bottom), tension crack (top left), view from the lake (top right) (photos by P. Bláha 2010)

Determining the tectonostratigraphic sequence of geological formations at the site was a very difficult task because the tectonic history of the area of interest is complicated and a range of lithofacies are present. During the Cretaceous, tectonics of Alpine style caused large-scale folding and thrusting accompanied by regional metamorphism. The present geological structure is the result of these complicated processes. Repeated episodes of folding took place during which individual tectonic slices were thrust up and more plastic rocks were accommodated between beds of rigid competent rock creating structural patterns of different orientation and widespread boudinage, secondary foliations, and small-scale folding throughout the whole formation. Exotic blocks of metabasite were also incorporated into various formations.

Since the Upper Cretaceous, uplift of the deformed massif has taken place leading to the formation of dome structures, a process that continues to the present day driven by neotectonic movements. The original geological structure of the whole lithotectonic complex has been affected by successive episodes of folding and deformation. Nappes were developed and thrust over one another. The last nappe, which was thrust over the lower complex folded earlier in the Cretaceous, provided the heat and imposed the pressure that was responsible for the final metamorphism of the rocks in the underlying massif. This configuration had an important influence on the pattern of metamorphism and metasomatism and the intensity of deformation which decreases with increasing depth in any section. This is because the distance from the overthrust heat source increases with increasing depth. This inverse relationship had an important effect on the engineering geology of the area.

In the area affected by construction work, a total of thirty tectonic dislocations were identified and twelve related systems of fractures were mapped and interpreted using background information (Fig. 3.3.3). Repeated episodes of movement along these faults created pronounced zones of weakness in the massif. These were selectively eroded to form the main topographic depressions in the present land surface. Radial fractures have created precipitous rock faces on the slopes, and systems of open fractures in limestone forming the marked ridges show incipient signs of karst phenomena.

In summary, the Centro Cuba PSHEP project lies in an area of complicated geology that has been strongly affected by Alpine style tectonics and metamorphism. The rocks have been severely weakened by faulting and fracturing that has controlled the pattern of erosion and deformation of the massif and tectonic movements are still taking place under the influence of neotectonic stresses. Karst is also developed in the fractured limestone of the Cretaceous massif (Fig. 2.3.13). The distribution of rock, semi-rock and the Quaternary cover also shows complicated patterns of deposition within the bedrock topography. All this contributes to the complexity of the hydrogeological regime. The first horizon is shallow with a combination of pore- and fissure-controlled permeability. The second horizon is deeper with circulation governed largely by fissure-controlled permeability with the confined water level and a high level of mineralization.

The area of the PSHEP continues to be seismically active with events once in a hundred years reaching up to the sixth degree of the MCS. Intense tropical weathering extends to a depth down to 40 m below surface. This further contributes to the complexity of the engineering-geological conditions that must be dealt with during construction. The strategy used for the initial survey and for engineering-geological mapping had to be planned taking account of these conditions, as well as the fact that it was an area difficult to access in which no detailed geological investigations had been carried out.

The coordinates of 42 profiles with a total length of 26,860 m were fixed before beginning the systematic geological field survey. Points spaced at 20 metres along each profile were surveyed. The natural exposures along each profile were mapped and described and ground-based geophysical surveys along most of the profiles were carried out. The information obtained was used to compile the engineering-geological map and to design the layout of exploratory boreholes and pits. The interpretation of the geology depended on combining all the information that could be obtained by geological mapping, and from the interpretation of aerial photographs, the results of geophysical surveys, and from logging cores recovered from boreholes. The morphostructural analysis assisted by the interpretation of aerial photographs played a pivotal part in the investigation. The results of the engineering-geological mapping, as well as the new information obtained from exploratory boreholes, down-hole logging, ground-based geophysics and by other applied methods were continuously evaluated. This enabled the plan of exploration to be adapted, as necessary, to take account of new information. As the extreme complexity of the area became apparent, the network of geological and geophysical profiles was progressively extended and the range of special measurements and tests made on boreholes, in adits and in the laboratory was modified.

During the four years of survey work carried out in stages up to the detailed design of the PSHEP, a total of 16,008 metres of exploratory drilling, 126 metres of pits, 399 metres of exploratory adits, and 805 metres of deep rock cuttings were completed. Observations and measurements at 1,585 points were documented, and 85,225 metres of profiles were geodetically surveyed and stabilized. The engineering-geological map covering a total area of 14 km^2 of the future PSHEP was compiled at a scale of 1:5 000 (reduced in Fig. 4.4.6). The guideline of the Czech Geological Bureau recommends 50 documented points per 1 km^2 for mapping at 1:5 000 scale in areas of complicated geology. In the PSHEP area 65 points per 1 km^2 were documented. In the 9 km^2 area of the construction site itself, up to 110 points per 1 km^2 were documented, which gives an indication of the complexity of engineering-geological conditions of this special area.

The most problematic dam sites are those underlain by sedimentary rocks, the strength and other important properties of which depend on the degree of cementation of the sedimentary particles and the composition of the cement. In the case of limestone in which fracturing and karst processes lead to enhanced porosity and permeability, there may be

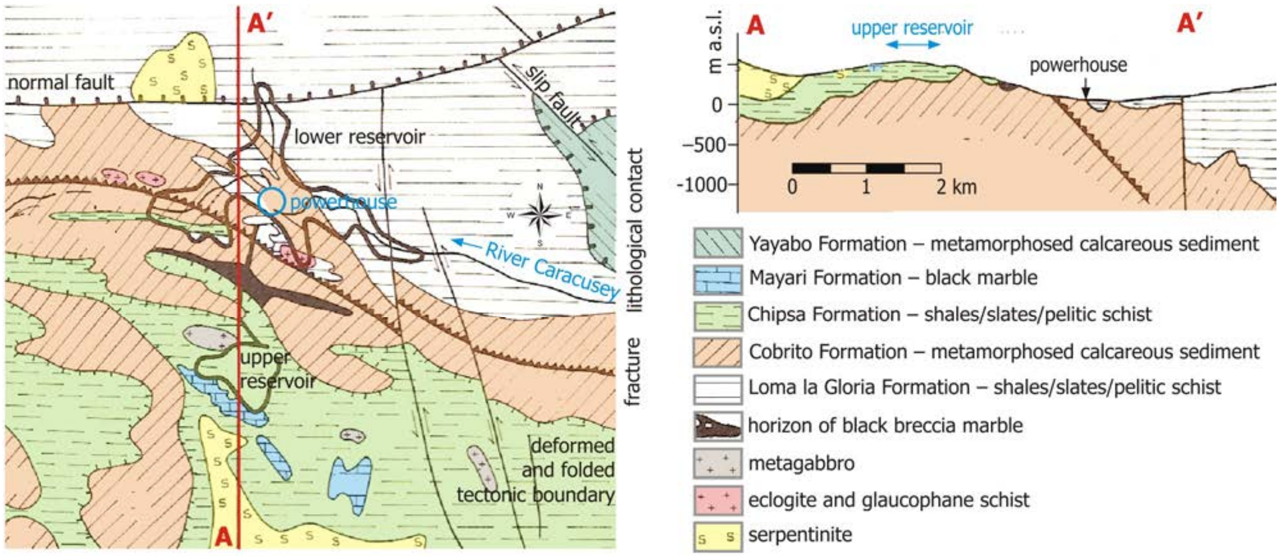


Fig. 4.4.6 Geological map and cross section showing the rocks and structures in the area of the Centro Cuba PSHEP

significant problems. Special attention must also be paid to fractured effusive rocks where rapid cooling results in abundant open cracks and scoriaceous beds that are usually of high permeability.

Even after the construction of a dam and its ancillary facilities in a complex area has begun, it is essential to continue monitoring artificial exposures, cuts and excavations to observe any changes that might be taking place. This allows understanding of the geological structure and its response to the new stress regime to be improved and warning of any potential risks given so that remedial action can be planned. Monitoring during construction is a standard procedure, even in areas with relatively simple geological structure, but much more detail and attention is required in areas of special complexity. These monitoring procedures add to the bank of engineering-geological knowledge that can be used in planning future projects safely and economically, allowing predictions to be made about the problems that might be encountered in analogous situations.

5 Hydrogeological Survey

The study of hydrogeological conditions usually forms an integral part of an engineering-geological survey. In certain countries where there is intensive development of karst phenomena (e.g., Cuba, Spain, the Balkan countries), hydrogeological conditions are the main factor limiting the construction of dams (Fortaleza, Corojo, Charco Redondo, Amistad Cubano - Búlgara, Foix, Guiamets, etc.). Apart from these special cases, there will be many dam sites with complicated hydrogeological conditions for which a detailed hydrogeological survey is required.

A surveyed area must be subdivided into aquifers and aquicludes and the individual bodies of groundwater identified. Based on this principle, the hydrogeological survey is undertaken in two stages:

- The first stage is concerned with the initial hydrogeological conditions before they have been changed by excavation and construction of the dam and ancillary facilities; and
- The second stage is concerned with the changes to the hydrogeological conditions caused by construction and operation of the dam and its ancillary facilities.

During the period of construction and before the dam is commissioned, it is necessary:

- To make an inventory of local sources of groundwater that may be affected by the operation of the dam;
- To identify potential sources of pollution (they may be initiated after the dam is commissioned), and to propose measures for their remediation or prevention;
- To assess the impact of upward pressure when excavating pits for construction (breaking up of the pit bottom);
- To estimate the flow of water into construction pits and the detrimental effects of pumping away the influx (destabilization of pit walls due to inflow pressures, possibility of suffosion); and

- To determine the mineralization and reactivity (aggressivity) of individual groundwater bodies in relation to the structures of the buildings.

For the period of operation, it is necessary:
- To assess the possibility of leakage of impounded water through rocks forming the foundations beneath the foot of the dam and through Quaternary sediments and the rock mass on the flanks of the dam;
- To assess the extent of possible water leakage through permeable zones created by shearing, fracture zones and faults, etc., at the dam site and in the reservoir area;
- To identify karst phenomena and assess their effects. This problem is of utmost importance;
- To determine the likelihood of mechanical and chemical suffosion;
- To assess the impact of the impounded backwaters and fluctuations of the water level in the reservoir on the groundwater regime in its vicinity;
- To determine how local sources of groundwater (of which an inventory will have been made) are affected by the new hydrological regime and how conditions for their exploitation have changed; and
- The efficiency of measures designed to prevent pollution from the identified sources.

The hydrogeological survey should be carried to a level that is adequate to plan the construction of cut-off walls, drainage and control systems, and to monitor the hydrogeological changes taking place during construction.

5.1 Preliminary Hydrogeological Survey

At the preparatory stage, a regional hydrogeological survey is made to enable comprehensive assessment of the hydrogeological conditions in the river basin in which the dam will be constructed so that the various options for the dam site can be assessed. The first stage is to review the existing archives of geological and hydrogeological data that will form the basis for determining the alternative scenarios available for exploiting the resources of the basin and planning a full hydrogeological survey in the field.

In mapping, emphasis is placed on the following tasks:
- Location of the natural and man-made sites where groundwater is extracted, channelled or monitored. These include springs, hydrogeological boreholes used for monitoring, wells, captation structures, agricultural drains, caves, and water-bearing joints, faults and fractures. Measurements of their yields, their pH and their temperature should be made and, if appropriate, partial chemical analysis to determine their chemical aggressivity;
- An assessment of the system of circulation and drainage of groundwater; and
- The preliminary assessment of the permeability of rocks and soils, especially with regard to the possibility of seepage through the basement beneath the foot of the dam and lateral seepage on the flanks of the dam and into the drainage of neighbouring valleys.

The regional hydrogeological study should provide sufficient information to enable the sections of the valley with the most suitable hydrogeological conditions for a dam site to be identified so that a decision can be taken.

By the stage of the preliminary survey for a planned design (economic feasibility), the site for the dam will have been chosen, or several sites will have been rated as equally suitable. The hydrogeological survey at this stage is therefore focused especially on the study of selected profiles and on critical sites in the backwater area of the future reservoir.

A hydrogeological survey across the selected dam profile is made in order to assess the effect of groundwater bodies during the stages of excavation and construction and, after completion, when the dam is commissioned. Special attention must be given to the impact of changes in the hydrological regime caused by the construction and the impounded backwater on the potential scale of losses by filtration and to the changes in hydrostatic pressure on the downstream and upstream faces of the dam.

The scope of the hydrogeological survey must be sufficient to provide all the data needed to develop a hydrogeological model of the studied area and to determine the hydraulic characteristics of individual groundwater bodies and confining units and their spatial distribution. The depth to which the survey is carried is generally limited by the presence of a sufficiently thick aquiclude, the coefficient of permeability of which is on the order of 10^{-7} ms^{-1} or less.

If a relatively impermeable layer is not found at a reasonably accessible depth, the following procedure is adopted:

- If permeability reduces with increasing depth, the general rule is to drill to a depth at which permeability is ten times lower than the permeability of rocks immediately in the foot of the dam foundations;
- If the permeability of the rocks does not decrease with increasing depth, in the case of concrete dams, boreholes will be extended to a depth equal to at least twice the height of the backwater (2 H), and in the case of embankment dams 1.0–1.5 H;
- In very difficult hydrogeological conditions (confined or saline groundwater bodies, karst, etc.), the survey depth may be even greater than those recommended above; and
- If water pressure tests are made, the borehole depth is usually considered as sufficient when the permeability of rocks at three successive levels matches the Jähde or Lugeon criterion of permeability.

The recommended upper limit of the hydrogeological survey will be set by the level of the highest aquifer in which a groundwater body will be formed after the reservoir is filled. During this process, changes in hydrodynamic pressure will take place and flow will occur. The extension of the survey onto slopes depends on the permeability of the rocks in the zone affected by the backwater and also on the maximum height reached by the backwater and the pattern of fluctuation (e.g., random or steady maximum level, etc.). The scope of the survey should also be in agreement with the calculated patterns of flow. The approximate scope of the hydrogeological survey can be determined using the following criteria:

- If the banks are composed of more or less impermeable or slightly permeable rocks, permeability measurements can be limited mainly to the zone of cover formations and weathered solid rocks (bedrock);

- If the rocks are of medium permeability, it is recommended that the survey is extended onto the surrounding slopes up to a height one to two times the height (depth) of the maximum level of the backwater (1–2 H);
- In highly permeable rocks, where there are concerns that losses by filtration and by-pass or seepage into neighbouring valleys will be large, or where saturation and reduction of slope stability could occur, the surveyed area could be several times larger (reaching to a height of 4–6 H on the surrounding slopes); and
- In karst areas, it is necessary to take account of the problems that affect each individual site and there are no limitations set on the scope of the survey that would be required.

Based on experience, it is necessary to locate the peripheral boreholes on slopes where they will encounter the groundwater table during dry seasons at the same elevation as the predicted maximum level of the backwater in the reservoir. For example, in the karst area of the Charco Redondo dam, boreholes on the alternative profiles selected were sited at a distance of 1–2 km from the lateral keying of the dam.

For a dam profile itself, or for each of the proposed alternative dam profiles, the following information is gathered during the hydrogeological survey carried out at the stage of project design:

- Preliminary measurements of rock permeability in the area affected by construction work;
- Identification and description of groundwater bodies, their inter-relationships and connections with surface streams;
- Determination of the properties of the groundwater and surface water, particularly their chemical aggressivity to concrete and reinforced concrete structures and their suitability for use in the production of concrete and grouting mixtures, and for the intended future uses of the reservoir water;
- Measurements of the depth of the groundwater table and, if possible, the fluctuations in groundwater level, at least during the period when the survey is being carried out; and
- Estimation of the dimensions and technical design for a grout curtain, if necessary.

So that the information required at this stage of the survey is complete, the following tasks must be undertaken:

- Laboratory measurements of soil permeability;
- In the case of the bedrocks, several slug tests in boreholes should be made, together with infiltration tests in pits and short-term pumping tests (depending on hydrogeological conditions) so that the hydraulic parameters of all the types of rocks can be determined;
- Water pressure tests should be made to determine how the permeability of the rock mass changes in relation to depth, and, if necessary, preliminary grouting can be carried out;
- The groundwater level in each borehole and the level at which it stabilizes after 24 hours should be measured;
- Additional measurements of the groundwater level in all boreholes should be made at least once a week during the period when drilling is being carried out; and

- Water samples should be collected from each groundwater body and from the main surface streams so that complete chemical analyses can be made to determine the aggressivity and to check for contamination from potential sources of pollution identified during the preliminary survey of the area.

In the backwater area, it is above all necessary to:
- Assess the permeability of the rock formations that form the flanks of the main valley and the divides so that the likelihood of escape by seepage from the water reservoir into adjacent drainages can be determined;
- Determine the depth of the groundwater table and the range of fluctuation in relation to potential seepage into neighbouring valleys;
- Identify groundwater bodies and the relationship between them and with surface streams;
- Assess the impact of the impounded backwater on the groundwater regime in the area affected by reservoir construction and any environmental impact this may have, including an assessment of possible disturbances of slope stability; and
- Identify potential sources of contamination and assess the risks of detrimental effects on the quality of groundwater and surface water in the catchment of the reservoir.

The information and measurements gathered during this survey work are recorded in the hydrogeological database of the project and presented on a hydrogeological map. The hydrogeological information can be combined with other engineering-geological observations on the same map sheet of the engineering-geological map or GIS layer, but in areas where the geology is moderately or very complicated, the hydrogeological map will be separate. Hydrogeological maps are usually compiled at a scale of 1:25 000 or 1:10 000 depending on the information required for design and other technical purposes. In recent years, increased emphasis has been placed on the quality of the environment, and it has become the practice to compile a hydroecological map to complement the hydrogeological map. A scale of 1:50 000 is best for hydroecological maps that should provide a more regional perspective on the future water reservoir and the surrounding area of the river basin in which it is located. The important features shown on these maps are the characteristics of the rock environment and the potential sources of pollution that could affect the groundwater in the catchment. The map will show the level of water saturation in the different types of rock, their state of weathering, and the configuration of tectonic structures in them (Michlíček, 1997). In most cases, the construction of a water reservoir has a positive impact on the quality of groundwater in the area, but the hydroecological map remains a valuable working tool, both during the planning stages of a dam project, and once the dam has been commissioned, allowing changes in the state of the groundwater before and after completion of the dam to be assessed.

A survey of the hydrogeological conditions in the backwater area of a reservoir involves the following tasks:
- At sites where losses of water by infiltration into neighbouring valleys could occur, a survey is carried out by drilling boreholes along profiles following the steepest line of the slope. The profiles are spaced one to two km apart and on each profile at least three boreholes are drilled. Usually, one or two of the three holes are drilled in a relatively impermeable layer. The recommended length of the holes should be double or triple the height of the backwater and, at the same time, the holes should penetrate at least as deep as the level within

which the groundwater level is expected to fluctuate so that the seasonal variations can be measured. For this purpose it is desirable to case about 50 % of the boreholes so that they can be used as wells for hydrogeological observation.

- In rocks with low permeability, hydraulic parameters are determined by slug tests. In very permeable rocks, it is advisable to carry out at least short-term pumping tests.
- If necessary, water pressure tests are also made in boreholes on slopes in the backwater area of a reservoir (e.g., Slezská Harta). And
- The basic measurements of the fluctuations of water level in boreholes are made in the same way as in the case of the survey for the dam site itself.

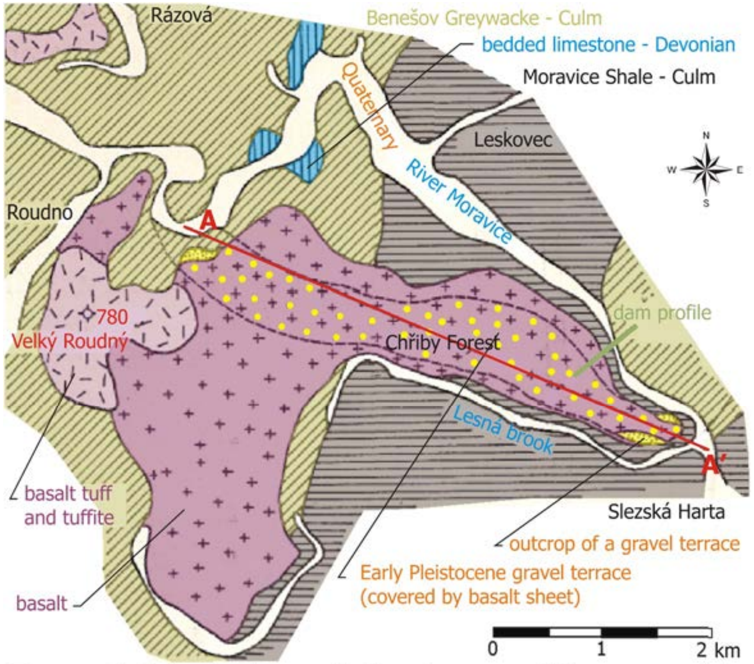


Figure 5.1.1 Geological sketch map of the area of the Slezská Harta dam. The position of the geological cross section in Fig. 5.1.2 is shown by the red line A-A' (adapted from Frejková, 1952)

One case in which it was necessary to solve serious hydrogeological problems at the preliminary stage of an engineering-geological survey was the dam on the River Moravice at Slezská Harta (1965 to 1968). Based on the original geological survey, the pre-Quaternary basement of the site is formed by rocks of flysch facies belonging to the Moravice and Benešov Beds of the Culm in which shale prevail over greywacke in the Moravice Beds, and greywacke prevail over shale in the Benešov Beds. The crucial formation that became the subject of the detailed survey is the sheet of basalt erupted from the Velký Roudný stratovolcano which completely covered the Early Pleistocene channel and valley of the River Moravice, including a few subsequent lava flows. The sheet reaches a width of up to 850 m and a thickness of up to 57 m (Fig.5.1.1). The latter stages of the survey demonstrated that this sheet of lava is divided into two flows separated from each other by a weathered scoriaceous horizon. The boundary between the two lava flows can be traced at a depth of 21 to 24 m below surface.

Between the Pleistocene channel and the present channel of the River Moravice, there is a long buried ridge of Culm. At the dam profile, the maximum height of the weathered mantle on the ridge lies at an elevation of 487 metres, i.e. 53 metres high above the level of the present river. The surface of the pre-Quaternary basement was found at an elevation of 480 metres, i.e. about 20 metres above the anticipated level of the future backwater (Fig. 3.5.2.). The flanks of the Culm ridge are covered by basalt blocks, which have sunk into the mantle of Culm rocks and moved downhill towards the present river channel. The cracks between the basalt blocks are filled with detrital material and are thus very permeable. This situation meant that there was a possibility that seepage from the reservoir through the buried Culm ridge and into the Early Pleistocene gravel could take place and it was necessary to establish the future maximum height of the backwater so that seepage could be prevented by a grout curtain or a comparable technical solution that would still be realistic in relation to the budget allocated for the project. The original design for a maximum backwater level at an elevation of 520 metres was therefore abandoned and a new maximum height for the backwater was set at 505 m a.s.l.

The subsequent survey work proved that the buried Culm ridge upstream reaches an altitude exceeding the height of the anticipated maximum backwater. Moving the dam profile to this site would have significantly reduced the costs of grouting work, but this was not a feasible alternative in terms of water management.

Another problem that had to be solved was that of seepage into the Pleistocene gravel terrace in the backwater area, where this gravel crop out at an elevation of 482 metres. Even though the survey work demonstrated that this gravel could be sealed by chemical grouting, it was

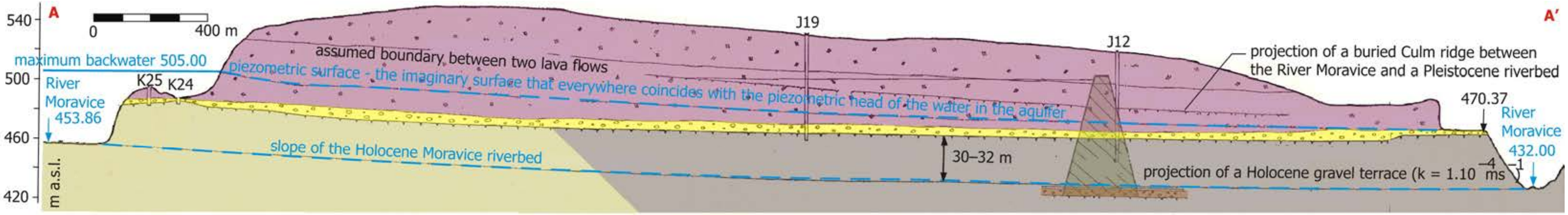


Figure 5.1.2 Geological cross section A-A' in the area between the River Moravice and the Lesná brook with the projected profile of the bed of the River Moravice and the piezometric surface of the groundwater in the basalt sheet governed by the height of water impounded in the reservoir

not necessary to take this precaution because the hydraulic calculations proved that the seepage would be negligible. Other calculations and measurements of the time dependence of groundwater level fluctuation in the basalt sheet on the amount of atmospheric precipitation (1980 to 1984) have also excluded the possibility of significant seepage into the Lesná brook adjacent to the dam profile, because the groundwater in the basalts there creates a natural barrier at an elevation of 510 metres, even during dry seasons (Fig. 3.5.2 and Fig. 5.1.2).

An example of hydrogeological mapping in little-explored and inaccessible terrain is the morphostructural analysis carried out in the course of engineering-geological and hydrogeological mapping for a dam and a pumped storage hydroelectric plant on the River Guanayara in Alturas de Trinidad in the Escambray Mountains (Fig. 5.1.3). The morphostructural analysis made there showed clearly that the structure of the area of interest and the river drainage pattern in the upper river basin were related. The alternation of soft, less resistant shale with resistant crystalline limestone and quartzite composing the mountains led to selective erosion and denudation that was amplified especially by lines of

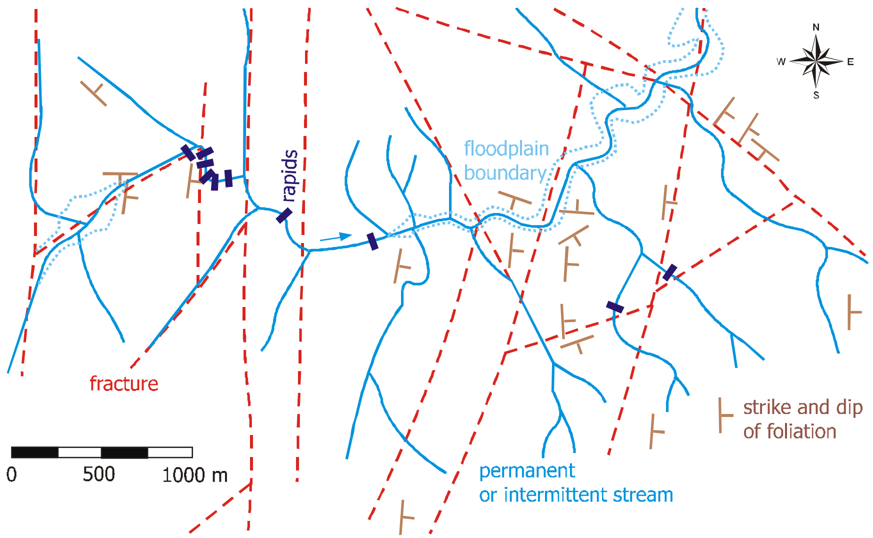


Figure 5.1.3 Sketch map showing the network of drainage in the upper basin of the River Guanayara in relation to the tectonic lineaments in the area (after Hrdý and Valeš, 1983)

tectonic weakness (Hrdý, Valeš, 1983). A similar relationship between geological structure and the pattern of rivers and streams is shown by the drainage of the River Genal depicted in Figure 3.3.2.

One of the hydrogeological problems encountered during the preliminary stage of a survey is the need to know the direction and velocity of flow of groundwater at the dam site and at sites where other buildings will be constructed, as well as in the backwater area. At this stage of the site investigation, the results of deep drilling are not yet available, so it is necessary to use all pre-existing information in combination with surface geophysics. A survey by the mise-a-la-masse method using a fixed electrode is an effective electrical procedure. An example of the application of this method in the investigation of the Josefův Důl dam site is illustrated in Figure 5.1.4. The direction and velocity of groundwater flow were interpreted using the measured difference in the direct-current potential field following salination of the water in a pit (V0) and after an appropriate period of time had elapsed (Vt). Using these measurements, two different directions of flow were distinguished. By coincidence both of them showed the same velocity of groundwater flow. The pattern of the potential contours was used to determine the flow in the direction 120 °. This method was preferred to that in which the direction of flow is given by connecting the centre of the elliptical field to the centre of the pit. In this case, the possibility that the fracture system controlling the flow of groundwater was most developed in the southwestern corner of the pit could not be ruled out.

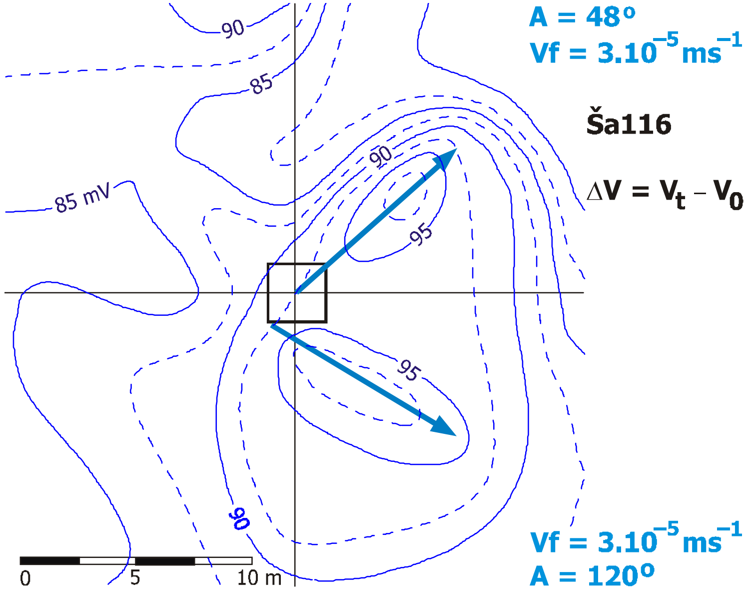


Figure 5.1.4 Plan showing the flow pattern of groundwater determined by geoelectrical measurements in the vicinity of a test pit at the Josefův Důl dam site

Figure 5.1.5 TV image captured showing stones thrown into the bottom

The cost of drilling and the need to use boreholes effectively for monitoring over a long period of time means that any programme of drilling must be planned carefully. When boreholes are to be used for long-term monitoring of groundwater level, the materials used to complete and case the boreholes must be carefully specified to prevent penetration by rainwater and it is important to cap the holes with strong and secure closures to prevent natural deterioration and damage by vandals. Figure 5.1.5 shows an example of inspection by television of a borehole in which it was not possible to measure the groundwater level. The record from the TV inspection clearly shows the borehole has been blocked with small stones and pieces of wood.

When monitoring of groundwater levels has been carried out over several years in a given area, it may happen that the response of the water level in certain boreholes to changes in the water regime in the rock mass is delayed, or that the level does not respond at all. If this happens after several years of measurement, the boreholes should be checked very carefully. In such cases, optical inspection of the borehole is recommended, ideally using a television camera. In Figure 5.1.6, two views of casing strings in observation boreholes are shown. In the right-hand part of the picture, the state of a plastic casing string in a 28-year-old borehole is shown: the perforation in the casing is more or less in its original state and is fully functional. In the left-hand part of the picture, there is a view of a steel casing string half a year after its installation in the borehole. It is almost impossible to see the perforation slots drilled in the casing. When a borehole is in this state, the extent to which the water level measured in the borehole corresponds with the real groundwater level can only be a matter of speculation. In places or during periods when there are rapid changes in groundwater level (GWL), the results of field measurements in such faulty boreholes will be unreliable. In most projects, the technical completion and equipment of hydrological boreholes is quite adequate, but there will always be a possibility that boreholes could be damaged or faulty.

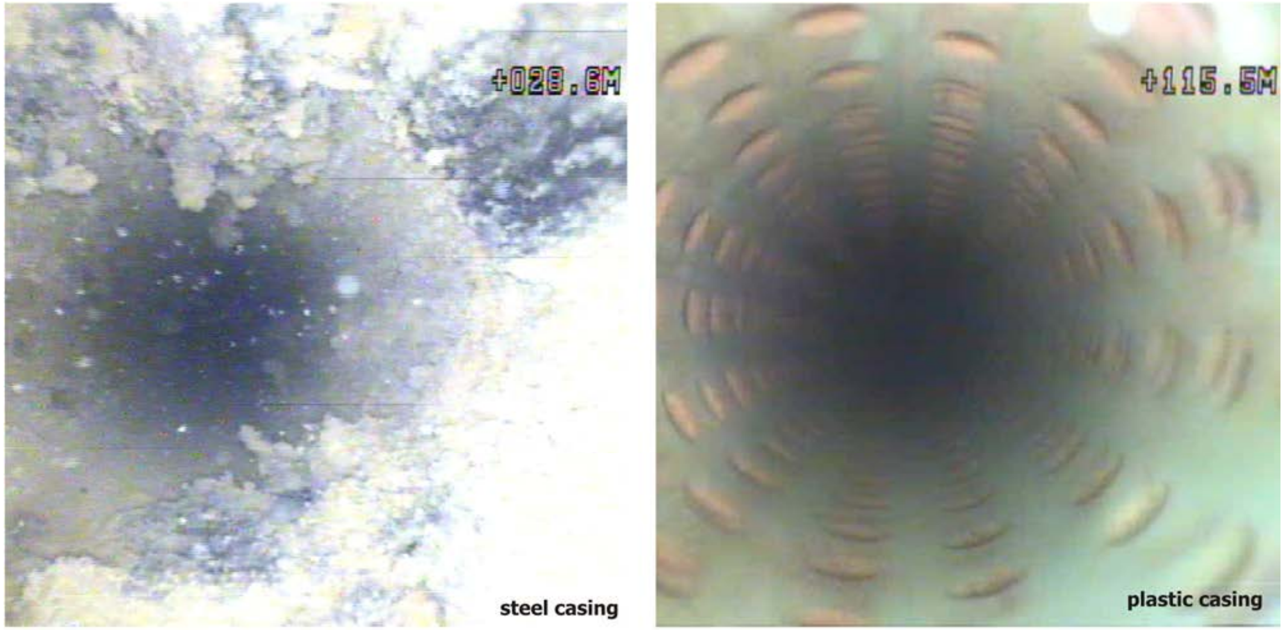


Figure 5.1.6 TV images captured during inspection of borehole casing

The final report on the hydrogeological survey involves a comprehensive evaluation of all the observations and measurements regardless of how they were made. It will be most important to show how the findings of the survey relate to the planned dam and its ancillary facilities. The final report must also state clearly any uncertainties in the data presented and offer proposals for gathering information data which will be required at the next stage of the engineering-geological survey.

5.2 Detailed Hydrogeological Survey

After the design of a project has been approved (feasibility), a detailed hydrogeological survey is carried out to provide the information required for the first stage of planning. After completing this, further surveys may be necessary to provide additional information required during the construction and commissioning of the dam and ancillary facilities.

5.2.1 Hydrogeological Survey for the Preliminary Design

After the final decision has been taken on the profile for the dam, the geometry of all aquifers and aquicludes must be defined and their coefficients of permeability determined. For this purpose, pumping tests are carried out in aquifers and slug tests are made in dry permeable layers. Water pressure tests are carried out on the rock mass to determine whether a grout curtain is needed and what its dimensions should be.

The number of pumping tests depends on the thickness of the aquifers, their geometry, homogeneity and permeability, and also on the effects they will have on the design and construction of the dam. If the thickness of a water-bearing aquifer is significant and there are variations in permeability, then it is necessary to conduct pumping tests at different levels. For example, in an engineering-geological survey for the Slezská Harta dam profile two different lithological units (gravel and basalts) were intersected in a borehole and pumping tests were carried out in each horizon. The tests showed that there was hydraulic continuity between the two water-bearing aquifers, and the hydraulic parameters of each were determined. Hydraulic continuity was demonstrated by analyzing the correlation between groundwater levels measured in observation boreholes in the separate horizons (Muzikář, 1984).

When making hydraulic calculations in rocks, the anisotropy is of great importance. Therefore, it is necessary to measure permeability in different directions, at least in the vertical and horizontal directions. As in other geological investigations, it is most convenient to carry out tests ***in situ***, no matter how perfect the data from laboratory tests may be. For the more accurate determination of hydraulic parameters and for the verification of the radius of influence, observation boreholes are drilled near pumping wells, usually in two profiles perpendicular to each other. These holes are used to measure changes in GWL during the pumping test. This provides the information necessary for the design of a drainage system or about drawdown of the GWL. The pumping tests that are carried out reflect the real scenario that will be created by the future drainage system.

In order to assess the need for a cut-off diaphragm wall and to determine the depth to which it must extend, water pressure and grouting tests are carried out along the dam profile. If the water pressure tests show that there are large-scale losses of water, it is necessary to carry out a very careful study of the subsoil beneath the planned dam profile and to discover the causes of the high permeability. Problems may be caused by faulting and pervasive fracturing, karst phenomena, etc.

It is necessary to determine the coefficient of permeability of each type of soils above the level of full groundwater saturation by carrying out infiltration tests in pits. The number of tests must be sufficient to enable evaluation of the permeability of the various horizons distinguished in the geological sections across the profile. They must also provide sufficient information to make a range of hydraulic calculations for both classical and numerical modelling. It is possible to make a tentative assessment of the coefficient of permeability on the basis of grain-size analyses. This is the best approach when dealing with Quaternary rocks of variable composition.

In order to make reliable predictions about the hydrogeological conditions that will be encountered during construction work and after the dam is commissioned, careful observations of the groundwater regime must be made. It is appropriate to conduct the measurements of

GWL on a weekly basis over at least one hydrological year, and then to compare these measurements against the record of atmospheric precipitation to determine the degree of influence. If another monitoring borehole is located in the same hydrogeological domain in the close vicinity of a dam, it is appropriate to compare the records. The results should also be assessed against the records of average precipitation over the long term. Boreholes for hydrogeological observation should be located not only along the dam profile itself, but also on the upstream and downstream sides and also at sites where there is a possibility that serious seepage might occur. For example, during the hydrological year 1980, and repeatedly from February 1983 to April 1984, measurements of groundwater level were made on the right bank of the Slezská Harta dam site up to a distance of 1.5 km from the lateral keying of the dam (Fig. 5.2.1). It is appropriate to carry out such measurements during the construction of a dam and after it is commissioned.

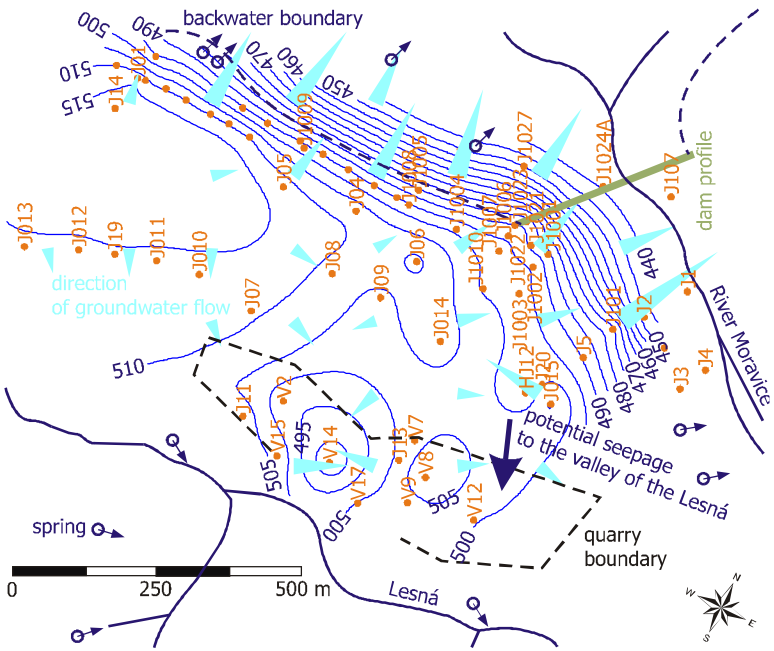


Figure 5.2.1 Map of the hydroisohypses on the piezometric surface on the right slope of the River Moravice showing the pattern of boreholes used to monitor the level of the groundwater

The pattern of the network of observation boreholes, the types of measurement and the frequency with which they are made depend on the specific features and complexity of the hydrogeological conditions at the dam site. The most comprehensive observations are carried out in those cases where karst is developed in limestone cropping out at the construction site or where there are a number of interconnected groundwater bodies confined in aquifers beneath the site. If soluble evaporitic rocks are present beneath a dam site, they can be the cause of major hydrological problems and must be surveyed with great care (e.g., salt domes at the Nurek dam site). Measurements of fluctuations in groundwater level and patterns of flow should also be complemented by periodic chemical analyses of groundwater samples (at least once during the dry and wet seasons).

If confined bodies of groundwater are present in a valley (e.g., borehole J 18 in the valley section of the original dam profile at Slezská Harta), it is necessary to determine their piezometric heads at different levels across the valley and to determine their origin. It is necessary to make detailed piezometric measurements of these confined waters especially at sites where deep foundation pits are designed. Without such information, it will not be possible to predict the effect of groundwater pressure on the stability of the walls and floor of the pit and avoid breaching. Cases of dam sites are known in which rocks were so disintegrated by artesian water penetrated by exploratory drilling that plans for construction of the dam had to be abandoned (e.g., the Poduzhemsk dam profile on the River Kem). Great difficulties arise when groundwater confined to a certain aquifer level are aggressive to concrete.

To determine the resistance of fine-grained soils to suffosion (piping), laboratory (pin-hole) tests are performed. Special hydrogeological investigations must also be carried out in cases where readily soluble evaporites (gypsum, halite, etc.) occur in the foundations of a dam or become active at depth beneath the foundations.

The most detailed hydrogeological survey must be carried out at sites where losses by percolation cannot be permitted for water-management reasons or where the stability of foundations could be disturbed by upward hydrodynamic pressure or chemical or mechanical suffosion.

A detailed survey in the future reservoir area is based on the same principles as a preliminary survey, but the amount of information required is much greater. Where there is a possibility that water will escape by infiltration into a neighbouring valley, the following tasks must be carried out:

- Permeable aquifers must be identified, their areas defined and their hydraulic parameters measured;
- The natural regime of groundwater in the area of concern, particularly in the watershed area, and its relation to the river and other groundwater bodies and surface streams must be investigated;
- If the possibility of seepage into a neighbouring valley is confirmed, the probable scale of seepage at different levels of backwater must be determined by calculation or by using a model;
- Predictions about the highest groundwater level (subterranean backwater) at different levels of filling of the reservoir, for example at the maximum level or at the longest-standing level, must be calculated or, if necessary, by designing a model; and
- Based on the observations made, a scheme of remedial measures must be proposed to prevent seepage, taking account of effects on the stability of the adjacent slopes and the dam itself. This will be based on a system of impermeable barriers, drains, etc.

At the Slezská Harta dam, the maximum level that would be reached by groundwater after the reservoir was filled and its impact on the backwater area were predicted by calculation as well as by modelling. Two scenarios were made - the one retaining the original natural conditions and the other with a number of installed sealing elements to prevent the escape of water by seepage through permeable horizons (Muzikář and Horský, 1984).

If the site chosen for the construction of a dam and reservoir lies in an area of karstified limestone, and the morphology of the valley and the structure of the limestone together create conditions that will enable leakage from the reservoir, the hydrogeological problems that have to be solved will often be challenging. Even in circumstances of this type, reservoirs have been designed and constructed, but often they were never successfully commissioned, or the cost of sealing permeable strata was unrealistically high. One of the cases in which the importance of hydrogeological factors was underestimated, is that of the Foix dam. This concrete gravity dam, 32 metres high, was completed in 1940 and was commissioned in the same year. The dam rests on karstified Cretaceous limestone. In the foreland of the dam, the limestone is unconformably overlain by Miocene sediments (Figs. 4.5.2 and 5.2.2). The water reservoir, intended to provide water for irrigation management, was designed to contain 6.2 million m^3 of impounded water.

Because the average annual inflow of water amounted to 8.9 million m^3, this was a realistic plan. However, due to the large amount of water continuously escaping into the permeable karst, the reservoir could never be filled up. Exploratory boreholes were later drilled to determine the causes and the scale of the seepage that was taking place. The groundwater table was intersected deep beneath the bed of the river and reservoir (1966–1967). Based on measurement of the changes in the level of water in the reservoir throughout the year, the relation between the reservoir water level and the amount of seepage was evaluated (Madurga and Doménech, 1957). Only 14 % of the water from the reservoir was actually utilized, due to the fact that the remaining 86 % of the annual inflow was lost, partly by evaporation, but mainly by seepage through the floor and sides of the reservoir. Already, by the end of the 1960s, the potential maximum volume of water that could be impounded in the reservoir had dropped from the 6.2 million m^3 originally planned to 5.5 million m^3 as the reservoir became silted up with bed-load sediments. Reservoir silting has continued and, today, after 67 years of operation, the remaining capacity of the reservoir is only 4.0 million m^3. One benefit of this process is that the accumulation of fine detritus has now partly sealed the floor of the reservoir.

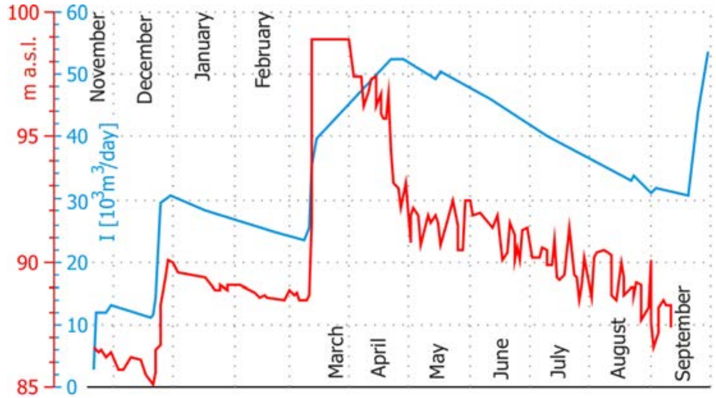


Fig. 5.2.2 Graph showing the relationship between the level of water in the reservoir behind the Foix dam and the rate of loss of water from the reservoir by seepage (adapted from Madurga and Domenech, 1957)

Today, the capacity of the reservoir is only 3 Mm^3, which is about 50 % of the volume originally designed for it. Since 1995, the reservoir has been used only for recreational purposes. This allows the level to be kept more or less steady. This example shows that technically difficult and expensive procedures for preventing seepage do not necessarily pay off. Sometimes natural processes will solve the problem after some years of operation.

If the preliminary hydrogeological survey shows that there is no possibility of seepage into neighbouring valleys and that the floor of the dam is impermeable, the detailed hydrogeological survey of the backwater area will be much simpler than in the circumstances described above. The scope of the survey will be reduced to the following tasks:

- To determine the natural regime of groundwater in the adjacent slopes and predict how this will change after the reservoir is filled, paying particular attention to conflicts of interest that might be caused, either during construction work or after the dam is commissioned. The facilities concerned could be public and private wells, roads, various scheduled monuments, recreational facilities, towns, villages, etc.;
- To predict the changes in the groundwater regime that will be caused by building activities and after the reservoir is commissioned, especially the destabilization of slopes, the development of subterranean processes leading to mechanical and chemical suffosion, surface erosion, abrasion, etc., especially in dams where a substantial fluctuation in the water level is anticipated; and

- To assess the potential risk of contamination of surface water and groundwater and the effect that the dam will have on land use in the flooded area and its vicinity.

In order to gather information about the likely changes in the groundwater regime and the problems these may cause, the following hydrogeological investigations are usually carried out:

- Detailed hydrogeological mapping that will enable the selection of sites and facilities suitable for monitoring the groundwater regime and, if necessary, surface waters in springs and wells, etc.; and
- Installation of hydrogeological boreholes along transverse profiles across areas where infiltration is anticipated. The distance between profiles is usually planned as follows:
  - o Water reservoirs near towns, industrial plants and other important objects: 300–500 metres;
  - o Villages, homesteads, recreational sites: 500–800 metres; and
  - o Agricultural areas: 1,000–1,500 metres.

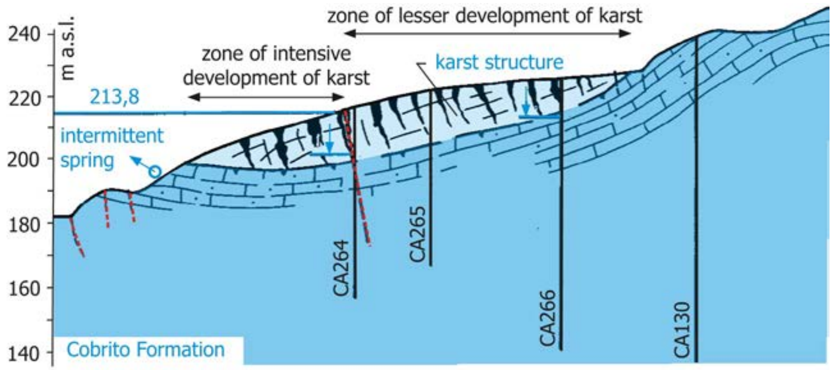


Figure 5.2.3 Geological cross section showing the scale of development of karst cavities in the limestones above the River Caracusey

It is usual practice to drill three holes along each profile and determine the tapped and standing groundwater level in each hole. In the case of deep boreholes, the "lithological" and "hydrological" features of the hole will be logged. Some of the boreholes can be used, as necessary, for pumping, slug or water pressure tests, and for making systematic measurements of the groundwater regime.

One interesting problem that was encountered in the original dam profile for the lower reservoir of a pumped storage hydroelectric plant on the River Caracusey was caused by a karst structure in the keying of the dam on the right bank. The karst phenomenon is located in the core of a synform in limestone deposited on underlying calcareous-graphitic shale (Fig. 5.2.3). The axis y trends NW–SE and dips 15 ° towards the future reservoir. On the axis of the dam the karst is not significantly developed but on the upstream side of the future dam it reaches to a depth greater than 15 m below the planned level of the

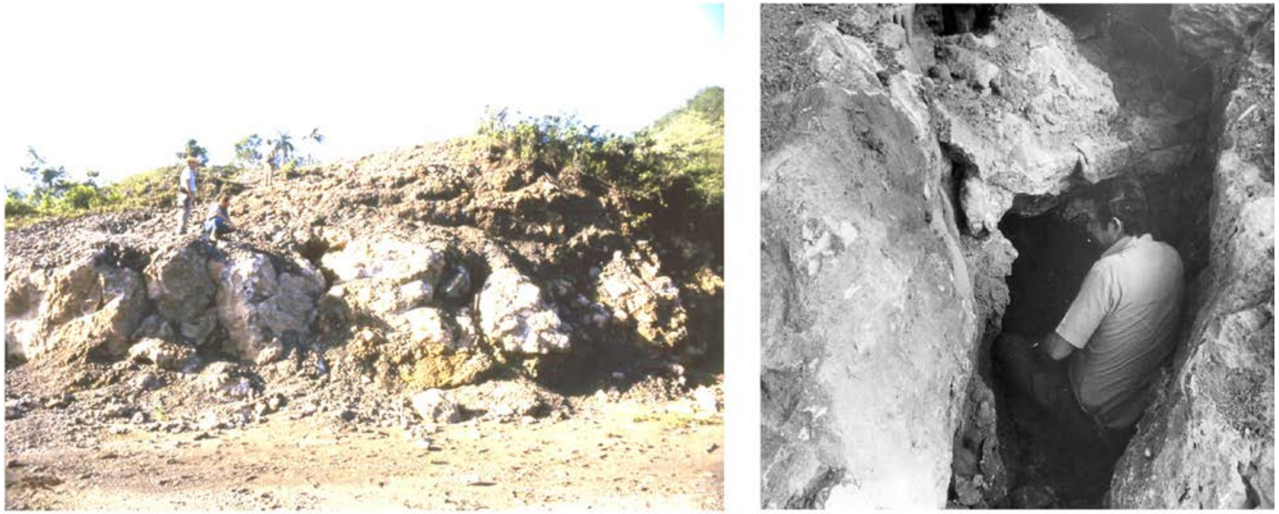


Fig. 5.2.4 View of the karst topography in the limestone at Caracusey showing a cavity large enough for a man to enter (photos by O. Horský - 1986)

backwater in the future reservoir. This created a major complication that would have caused a significant increase in the costs of construction. The geological survey also showed that individual limestone blocks were separated from each other by open cracks up to one meter wide (Fig. 5.2.4) and the blocks had slipped downhill over the underlying softer graphitic shale. It was possible for a man to descend to a depth of up to 5 m into this system of cracks, which was concealed by a rock mantle at the surface. Below this level, the cracks become narrower as they continue deeper into the slope.

Depending on the precipitation, the level of the groundwater at the site of the planned reservoir fluctuated between 10 to 15 m below the ground surface. During periods of high precipitation, there is rapid recharge and the water level quickly rises. This is followed by a period when the water level falls very slowly. This behaviour shows that the cracks and fractures are not completely interconnected and there is a barrier to outflow. In order to understand the behaviour of the groundwater, tests were carried out using fluorescein dye to trace the flow of water through the fracture system. These proved the hydrological continuity of the karst fissure system and demonstrated that moderate flow upstream into the valley was taking place. For these reasons, an alternative profile for the dam was proposed downstream, and subsequent work proved that this decision was the best.

5.2.2 Hydrogeological Survey for the Detailed Design

Once the decision has been taken to proceed with the construction of a dam on a definite site, it is necessary to determine in detail the hydraulic parameters that will govern the design of the systems of sealing elements and drainage in the immediate area of the dam and the important ancillary structures. The inflows of groundwater into the construction pits and the measures required to drain them and protect the stability of their walls from the effects of water pressure, suffosion, etc., must be determined. If necessary, suitable sources of water for the production of cement and concrete mixtures and for drinking must be identified. Detailed tests must be carried out to determine the chemical aggressivity of the groundwater to concrete and reinforced concrete structures. At this stage of the survey, a plan for the network of boreholes required to monitor the groundwater regime during the work of construction and after completion of the dam should also be proposed. If potential loss of water by seepage through the rocks at the dam site and around the banks of the reservoir was identified as a problem at the previous stage of the survey, detailed measures to prevent the escape of water by seepage must now be designed.

In some projects, the potential for leakage of water from future reservoirs was recognized and the necessary measures were proposed. However, it was not possible to fill the reservoirs up to the planned level after the dams were commissioned. In such cases the dams and reservoirs could not be operated at their full capacity for their designed purposes of water management or power production. The example of the Foix dam has been described above (Figs. 4.4.2 and 5.2.2); another similar case is that of the Guiamets dam on the River Asmat (Fig. 5.2.5). There, a 46-metre high concrete gravity dam was constructed on Triassic limestone and dolomite in 1983. Longitudinal geological sections through the banks of the reservoir and a schematic map of the dam and the adjacent part of the backwater area are shown in Figure 5.2.6. The volume of water impounded at an elevation of 184.87 metres was planned to be 9.7 million m^3, but this figure was not achieved. More than 80 % of the impounded water was being lost through seepage and only 20 % of reservoir water was utilized. In 1991, with the level of the backwater at an elevation of 175.60 m, the rate of water seepage reached up to 400 l/s and the losses were calculated to reach in excess of 1,000 l/s for a maximum backwater elevation of 184.87 m. As a result, in 1994, the Czech team was asked to reassess the geology of the dam site and prepare a proposal for reducing the losses by seepage. The assessment led to the following observations:

- The water reservoir and dam are sited on Triassic limestone and dolomite (Muschelkalk), overlying recrystallized claystone and variegated sandstone (Buntsandstein);
- In the area of the dam and in its foreland, the beds dip 20–30° to the SW, i.e. obliquely beneath the dam and towards its keying on the left bank;
- Karstification is governed by zones of intense tectonic fracturing that provides the permeability along which water circulates and causes chemical erosion of the limestone;
- At the level within which the groundwater fluctuated, depending on the amount of precipitation and the levels of water in the river and in the reservoir, water was penetrating the banks of the reservoir and escaping through karst structures;
- At the contact between the limestone and dolomite, there are wide open cracks and cavities that have formed as a result of tectonic fracturing combined with supergene oxidation and chemical solution by groundwater;
- The zone of stress relief accompanied by a higher degree of jointing and weathering reaches to a depth of 29 to 32 metres on the surrounding slopes;

Figure 5.2.5 View of the Guiamets dam in the Ebro Basin, Spain, showing grouting in progress on the left flank of the dam behind the abutment (photos by O. Horský - 1994)

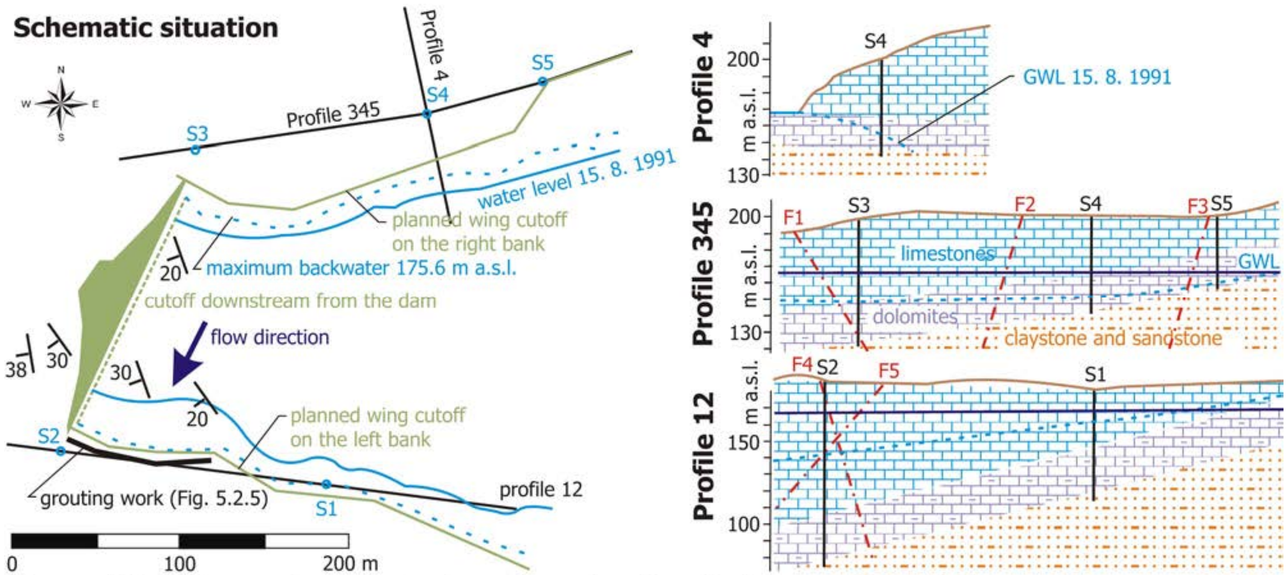


Figure 5.2.6 Plan and geological sections showing the layout of the Guiamets dam

- The underlying recrystallized claystone and variegated sandstone are slightly permeable to impermeable;
- The level of the groundwater in all of the five boreholes that were drilled is found to be lower than the height of the water in the reservoir. This indicates that there is a hydraulic gradient along which water is being lost into the slopes and beneath the dam itself; and
- It was not possible to determine exactly the losses of water taking place along specific zones of seepage.

A general idea of the overall losses could be determined only by calculation based on the known amount of water arriving and the known volume of water in the reservoir. Actual measurements of losses were made with success only at some places on the slopes on the downstream side of the dam and in service tunnels within the body of the dam.

Taking into account the observations described above, it was recommended that geophysical methods should be used to locate the permeable tectonically weakened zones in the rock mass. These zones govern the development of karst features such as caves, pipes and cavities through which most of the losses of water take place. A programme of drilling and grouting was also proposed in the area of keying of the dam on the left bank. This was designed to reduce losses of water from this area where the biggest rate of seepage was detected, as shown in Profile 12.

All the boreholes were systematically logged. In addition to lithological descriptions of the stratigraphy intersected by the boreholes, attention was paid specifically to fracturing in the rock mass. A full range of methods was used to log the holes, including measurements of hydrogeological conditions (so-called hydrogeological logging). These measurements were made to warn the contracting companies about the places where an increased consumption of cement could be expected in grouting work.

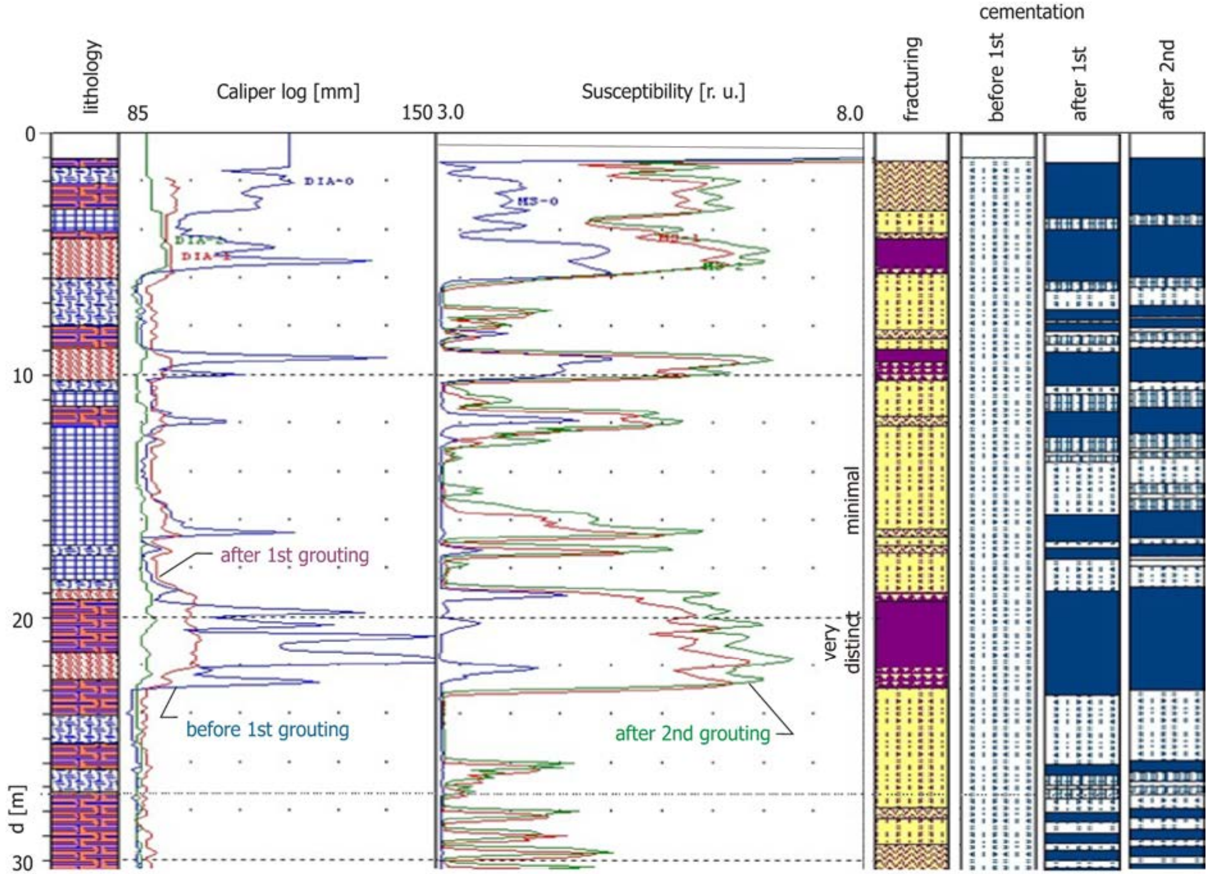


Figure 5.2.7 Logs of magnetic susceptibility demonstrating the progressive improvement in cementation of permeable horizons as a result of two successive stages of grouting (after Kořalka and Procházka, 1996)

After the first cementation, the borehole was re-logged to determine how much the rock fracturing and the hydrogeological regime had changed. If the cementing work proved inadequate, it was repeated again. After completion the borehole was always re-logged. Once it had been shown that the rate of seepage into the borehole was low, it was possible to move the grouting rig to another borehole.

Logging specialists from the company Aquatest Praha designed the optimum combination of methods that ensured that the grouted boreholes could be logged rapidly and thus also cheaply. The procedure used was the measurement of the magnetic susceptibility of the rock mass (Fig. 5.2.7). It was discovered that the hardened cement mixture in cavities and open cracks increases the susceptibility of the rock mass in the vicinity of the borehole. Magnetic susceptibility measurements were combined with hydrogeological logging to determine how large the rates of seepage into the borehole were. Taking the example of borehole 13lb in Figure 5.2.7, after the first grouting, seepage dropped from an initial value of 15,000 l/day to a value of 12,000 l/day, and after a second grouting it subsequently dropped to 500 l/day.

The picture of caliper log data clearly shows how successive grouting was successful in the remediation of cavities in the borehole wall. After the second grouting, the diameter of the borehole in the cavities intersected was already equal to the cutting diameter of the drill bit. Simultaneously, the magnetic susceptibility of the rock mass was rising. A striking difference between the measurements made before and after the first grouting is visible (blue and violet curves). Yet, some cracks remained open, which was indicated by the high losses of water during down-hole logging tests, and also by high rates of seepage that were calculated. Only the second stage of cementation fully sealed the cracks, even though it had little effect on the other properties of the rock mass. The last two columns depict the amount of cement that was injected into the rock based on the logged measurements. Detailed comparison of the curves for magnetic susceptibility at the successive stages of cementation shows that, in the third logging made after the second stage of cementation, a moderate error in the determination of depth had been introduced. It appears that the third curve should have been adjusted somewhat lower. This, however, is only a minor error in an otherwise effective procedure that was later used at other sites to check the effectiveness of cementation in boreholes.

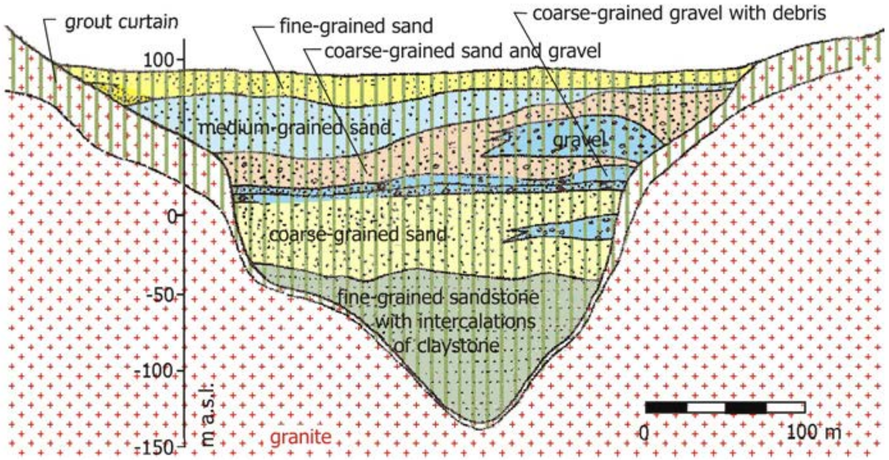


Figure 5.2.8 Geological section across the Nile valley at the site of the Aswan dam showing the grout curtain injected into the sequence of permeable alluvial sediments filling the incised channel (adapted from Yague, 2000)

Figure 5.2.9 View of the Aswan High Dam, (http://byebyenet.com /Egypt/Aswan-Abu Simbel/Aswan HighDam Egypt.jpg)

After the completion of grouting work in places where the seepage was largest, especially in the foreland of the keying on the left bank of the dam, overall seepage was reduced by 30 to 40 %. In the subsequent extremely dry period between 1997 and 2006, the reservoir could not be filled to more than 38 % of its designed capacity. Only in the course of the first half of 2007, thanks to heavy precipitation, it was possible to fill the reservoir to 62 % of the planned operating volume. These results show that there is still some way to go before the water reservoir can be fully utilized according to the original plans. The operators are aware of this, and therefore further grouting operations are being considered. The difficulties experienced during the commissioning of this reservoir are a good example of what can happen if the initial engineering-geological and hydrogeological surveys are not carried out at the level of detail required to design the technical solutions required to prevent seepage. In this case it was necessary to carry out a survey after completion of the dam and propose an expensive programme of remedial work.

The example from Spain that has been given above shows how failure to carry out a detailed hydrogeological survey at the early stages of the project can lead to serious consequences later. In this case large-scale seepage prevented the dam and reservoir from being used for its planned purpose. Of course, it is necessary to state that conditions of zero permeability cannot be achieved in any reservoir, because absolute impermeability does not exist. If seepage is reduced to acceptable values and captured by a system of drainage, this is not a hindrance, because it is necessary to ensure that at least a minimum rate of flow is permanently maintained in the river below a dam, as set by the rules of reservoir operation. A good example of an effective large-scale plan to limit the permeability in the basement of a dam is the 111-metre-high, 980-metre-wide Aswan dam on the River Nile with a crest 3,600-metre-long. Locally, in the basement of this dam named Saad-al-Aali, there is a sequence of river sediments up to 235-metre-thick (Figs. 5.2.8 and 5.2.9) composed of sand and gravel of various grain sizes, with claystone and fine-grained sandstone forming the lowest beds. A grout curtain reaching a depth of 250 metres was constructed to prevent seepage. It consists of fifteen rows of grout holes. The rows are spaced from 2 to 6 metres apart and the individual holes are spaced two metres apart. The total length of grout holes reached 321,000 metres, and the volume of grout mixture used was 1,650,000 m^3. The permeability through the dam basement was thus reduced to a value of 3.10^{-6} m/s.

The examples above show that the underestimation of the hydrogeological survey may result in failure to meet the main requirement laid on the dam, i.e. the impoundment of water in a reservoir, whether permanent or temporary. It is therefore necessary to study very carefully not only the dam site itself, but also the future backwater area in relation to permeability, focusing particularly on the banks and adjacent slopes of the water reservoir. To reduce the amount of permeability at the dam site or in the backwater area, it is therefore necessary to propose appropriate remedial measures and evaluate them economically especially from the point of view of whether the amount of necessary financial costs of remediation is in compliance with the future purpose and use of the reservoir. The economic consideration must also include the costs of the maintenance of such remedial measures, such as the periodical control of the functionality of the grout curtain or its necessary after-grouting, but also the maintenance of remedial measures in the area of the reservoir.

6 Geophysical Surveys

Geophysics is the science concerned with natural and artificial physical fields. The scope of engineering geophysics covers the survey of construction sites in order to determine the distribution and characteristics of different types of rocks and structures, both in the subsurface and in the immediate vicinity of a designed construction. The appropriate choice of geophysical methods and the way in which they are used for the purpose of a survey depends on an understanding of the geological environment in question and the type of information that can be obtained by using the chosen methods. The evaluation of the results will depend on the physical theory on which the survey methods are based combined with relevant geological information obtained from other sources. The ideal circumstances are those in which there is cooperation between the engineering geologist, the geophysicist and other specialists throughout all stages of a planned survey. This applies both to the preparatory stage of the work, to the procedure of field measurement and, in particular, to the geological interpretation of the results obtained from the geophysical survey.

The interpretation of measured values in relation to other characteristics and parameters obtained from an engineering-geological survey enables a structural model of a surveyed area to be created. The combination of expertise contributed by the team should ensure that the model reflects the reality of the geological situation as far as possible. Due to the heterogeneity and structural complexity of the natural environment, no geophysical methods can provide a perfect picture of the geology of a particular site. The satisfactory outcome of a survey thus depends on an appropriate choice of the methods to be used and on the reconciliation of the theoretical model with the real geological circumstances. Ultimately, this depends on the experience of the geophysicist (and the engineering geologist) responsible for the interpretation of the results.

Geophysical methods always provide some basic information about the geological structure of a studied area. In some cases this information is precise and dependable, but in other cases there will be uncertainties. This may be because the geology of the bedrocks in the vicinity of a construction is complex, or because financial constraints limit the scope of the survey undertaken. Even if only general information is provided by a survey, this can be used to guide the choice of procedures (boreholes, pits, etc.) used for direct investigation of the site at subsequent stages of the survey. Information obtained by geophysical methods will always be useful because unnecessary field work can be avoided, and optimum sites for detailed investigation can be chosen in the shortest possible time. In this way, the appropriate use of geophysical methods at an early stage can improve the quality and efficiency of the entire engineering-geological survey.

The primary requirement of an engineering-geological survey for a dam site is that it provides reliable information about the composition and engineering properties of the bedrock, subject to the constraints imposed by economic factors and the resources available. Bearing this in mind, it is necessary to search all pre-existing sources of information about the area that will contribute to the safe design, construction and operation of the planned dam. Mistakes at the initial stages of a survey can have serious implications for later stages of work and remedial measures may prove demanding in both technical and financial terms. Geophysics, as a scientific discipline, is capable of yielding a great amount of useful information within reasonable economic limits at all stages of a survey. In many cases it provides information that would either not be available or which would only be obtainable using much more expensive methods.

The use of geophysical methods in surveys for hydrotechnical structures can be considered from two points of view. The first is the potential of individual geophysical methods to provide information based on the physical principle involved. The second is the capacity of a selected method to provide answers to the questions posed by a particular engineering-geological survey. In giving an account of the use of geophysical methods to survey dam sites, it is necessary to choose which of these two approaches to take. Each approach has its advantages and disadvantages. To provide a comprehensive overview using both approaches is not feasible. A third option is to take the unconventional approach which we will use in this book. The assumption is made that the reader has a basic knowledge of the principles on which geophysical methods are based and of the requirements governing engineering-geological surveys for dam sites. With this in mind, only a brief summary will be given of the geophysical methods commonly used by an engineering geologist for a geophysical survey. The main part of the book will be devoted to the specific applications of selected geophysical methods, unconventional approaches to their application and the interpretation of the results obtained, and the history of the use of geophysics to solve specific engineering-geological problems.

6.1 Tasks of a Geophysical Survey

A geophysical survey for a water-retaining structure can provide information about the following:

- Geological composition and structure at the dam site;
- Geological composition and structure in the reservoir area;
- Resources of construction materials;
- Conditions at the sites of ancillary buildings and services (roads, spillways, workshops, etc.);
- Changes in conditions on site during the survey and during construction; and
- Stability of conditions during operation of the facilities.

At the initial stages of a survey, it is unlikely that the answers to all these questions will be provided. It is natural to place emphasis on acquiring the maximum amount of information about the dam site itself. At the early stages, remote sensing methods can be used with advantage to characterize the topography, hydrology, vegetation and underlying structure of the site, especially in cases where high resolution stereoscopic satellite images covering a range of spectral bands are available. Also, data obtained by aerial geophysical methods will be of great value, particularly in surveying remote and poorly explored areas. Obviously, the best results will be obtained by a combination of several methods using modern GIS systems.

When investigating a dam site, the first objective must be to identify the different types of rocks that make up the area and determine their distribution in relation to the underlying structure. Zones of weakness in the rock mass and the tectonic factors responsible for them must be delineated. Finally, at this stage, it is necessary to determine the physical and mechanical properties of the rocks that ultimately govern the design of the water-retaining structure. The permeability of the rock mass must also be measured and a decision taken on the position and depth of a grout curtain, if required.

An example of a geophysical survey made to provide a preliminary orientation for an engineering-geological survey of a dam site is that from the left side of profile P6 on the River Genal (Fig. 6.1.1). Geophysical sounding was carried out using shallow seismic refraction (SSR) and vertical electrical sounding (VES). The results from SSR were interpreted using the method of critical distances and also by the method of velocity of propagation, which permits the increase of velocity and its changes below the refraction horizon to be determined. Resistivity profiling was also carried out using two patterns of electrode spacing. The Wenner configuration with a depth of penetration of 10 metres and the Schlumberger configuration with a depth of penetration of about 30 metres were chosen. Symmetrical resistivity profiling was complemented by the use of the very low frequency (VLF) method and by measuring the pattern of the Earth's magnetic field (T_a). In addition to these profiling methods, the velocity of the longitudinal waves on the refraction horizon was also obtained by interpretation of the SSR measurements.

As noted above, the interpretation of the results obtained from the geophysical survey in the example given can be considered from two points of view. In the first case, the geophysical measurements can be

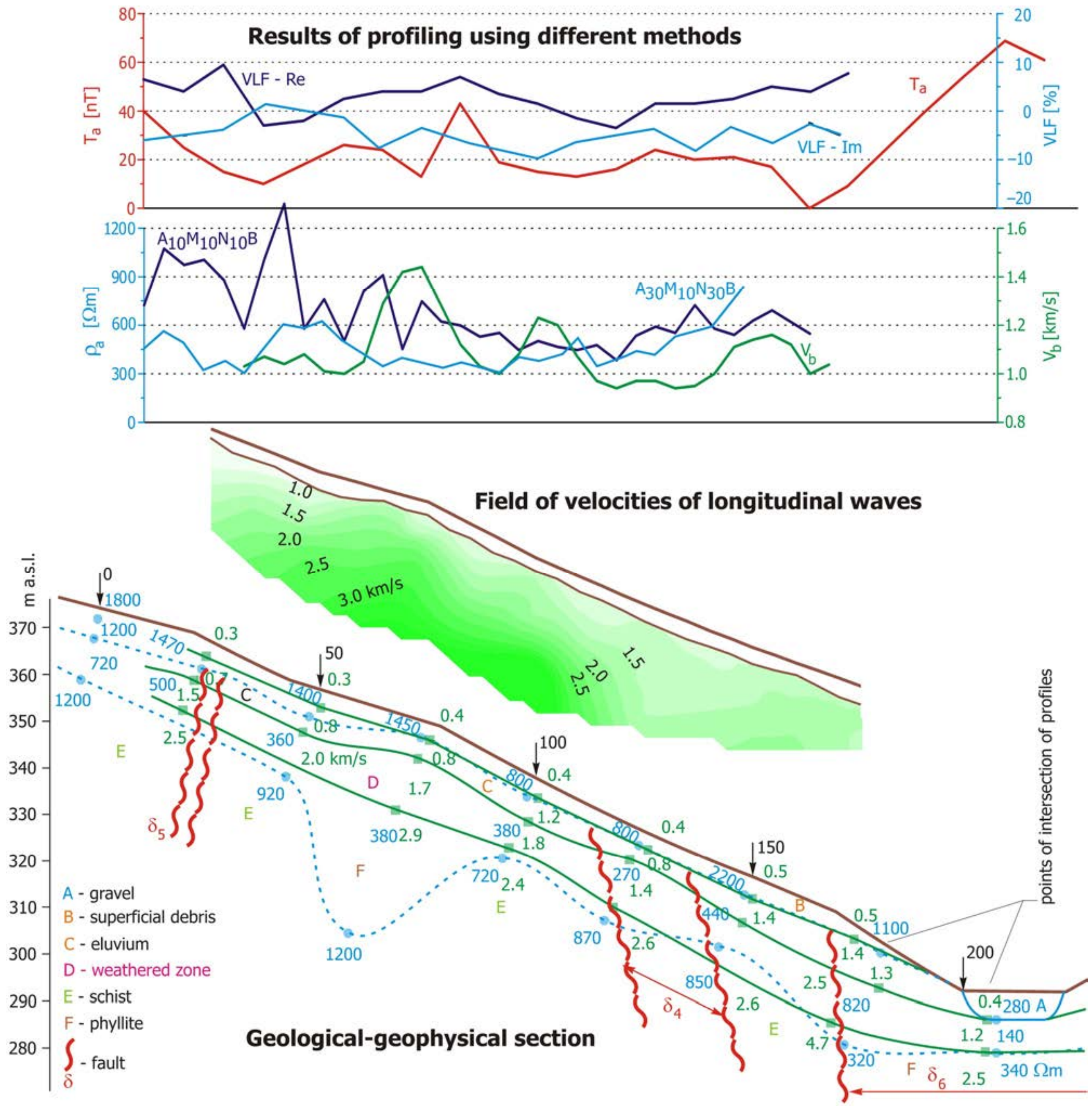


Fig. 6.1.1 Genal dam profile, a survey of the left slope

placed in their context by the geologist in order to answer questions about the underlying geology and structure. The second approach is for the geophysicist to describe the situation in purely objective terms by defining the variation of physical properties within the rock mass. Both these approaches are valid and it is difficult to say which approach is better. Of course, the ideal strategy is to combine the two approaches, so that all the measured variations in physical properties can be given a rational geological explanation. An intrinsic part of this process is to explain the logic on which the geological interpretation of the geophysical results is based. If there are uncertainties or ambiguities, attention must also be drawn to these.

In the example given, the geophysical measurements were used as a basis for dividing the rock mass along the surveyed profile into six quasi-homogeneous units. Units A and B represent the Quaternary cover, the remaining units being part of the bedrock forming the Betic Cordillera. Unit A is identified as gravel, and Unit B is a cover layer of silty clay or silty clay with debris. Unit C is the upper part of the eluvium and Unit D the lower part of eluvium on the bedrock. Units E and F are the practically unweathered bedrocks. In this case, Unit E corresponds to schist and Unit F to phyllite. In geoelectrical terms, layer B is characterized as the layer with the greatest range of resistivity. This indicates not only the variability of its geological composition but, in particular, its high degree of aeration. It can be concluded that this not-too-thick near-surface layer is formed mostly of debris. In the valley bottom, a layer of gravel is identified in a single VES.

Layer C cannot be distinguished geoelectrically from layer D. These are the eluvium and the parts of the rock mass distinctly affected by weathering processes. In absolute terms, the changes in resistivity are not as great as in layer B, but relative changes are higher and reach a value of 5.9, which is more than twice that in the layer of surface debris. The content of clay minerals in the rock has the greatest effect on the value of resistivity. It does not matter whether these minerals originated *in situ*, or were carried into place as a secondary deposit by groundwater action. The moisture content of specific parts of the rock mass also has a significant effect on the value of resistivity and a conductive network can be created by a combination of moisture and the clay minerals present in the rock. This can reduce the resistivity values of the rock mass by one or more orders of magnitude. Using resistivity criteria, these units can be characterized as a layer of variably weathered rocks with different admixtures of clay minerals and moisture contents.

The resistivity of the lowest layer, the bedrock (Units E + F), appears to be very variable. However the fact that it consists of two different rock types must be taken into account. In this case, along the surveyed profile, the resistivities of the phyllite range between 320 and 380 Ωm, whereas the resistivities of the schist vary between 720 and 1,200 Ωm. This explains why the resistivity of the bedrock as a whole spans such a wide range.

Seismic measurements enable a more detailed subdivision of the near-surface layers on both slopes of the surveyed valley. The velocities of longitudinal waves in Unit B match what would be predicted for the velocities of longitudinal waves in silty clay and debris. The same applies to the velocities of seismic waves in the layer of eluvium (Unit C).

It is of great importance for building work that both these layers should be excavated easily by machinery. The velocities of longitudinal waves in the transitional zone (D) appear low and do not correspond with those for weak rocks. We must, however, take into account that these velocities were determined by the method of critical distances (the refraction point method), which determines the seismic velocity on the surface of the given layer. In this case, the surface lies at a depth of about 10 metres where the layer has properties close to those of eluvium. Another reason for the decrease in velocity is the low moisture content of the rock mass on the valley slopes. The bedrock has velocities that correspond to materials in this state. The decrease down to a value of 2.4 km/s results from the weakness of the rock in places where the mass has been fractured. It is important to note from detailed observation that higher velocities were determined where measurements were carried out in a direction perpendicular to the monitored profile. In this case, seismic waves did not have to penetrate the whole belt of fractured rock, but could "accelerate" in beds of relatively intact rock either directly along the fracture or by "propagation around" the fracture through the surrounding intact mass.

Magnetometric measurements defined a "positive" anomaly above the floodplain of the River Genal. It is likely that this anomaly originates in the gravel and sand in the floodplain. The river sediments obviously contain pebbles of "exotic" rock and detrital minerals that were carried into the area of interest by the River Genal from the upper part of its drainage. No other magnetic anomalies were detected on the slopes of the Genal along any of the four studied profiles. The area surveyed extends for about 12 km and magnetic anomalies were not detected, even in areas of faulting and fracturing. During field trips no exposures of rocks with high magnetic susceptibility were encountered.

In the reservoir area, it is necessary to pay attention primarily to the stability of the slopes of the future reservoir and to identify sites where water could potentially leak from the reservoir. To assess the scale of the first problem, it is necessary to carry out a survey of all active and fossil slope failures and identify the sites that might be prone to failure as a result of construction and operation of the proposed dam and reservoir. It is necessary to take into account that slope failures can be triggered by erosion of the banks in the future reservoir or by other physical processes that will affect the surrounding geology after the reservoir fills up. In this context, it is necessary to establish where there are large thicknesses of cover formations and places where the rock mass has been subject to intense tectonic deformation and fracturing. To assess the scale of the second problem, areas where highly permeable rocks crop out within the footprint of the proposed reservoir and dam must be identified. Permeability may be due to the intrinsic properties of the rocks or to the effects of tectonism or to a combination of both these factors. The survey carried out will also involve the selection of suitable sites for possible saddle dams and dykes in the reservoir of a future reservoir.

In cases where there is expected to be a large fluctuation of the water level in a reservoir, for instance when a dam is intended to supply water for irrigation in arid regions or other circumstances in which abrupt fluctuations are predicted, it is particularly important to have a good understanding of the geological structure of the slopes surrounding the dam and reservoir. A large change in the water level of a reservoir, especially if this takes place rapidly, is likely to have an adverse effect on the stability of the surrounding slopes that is likely to trigger landslides and rock falls. Figure 6.1.2 shows a section through the Mingchukur landslide at the Charvak reservoir. The height of the rockfill dam is 168 metres and the annual fluctuation in water level is in excess of 80 metres. The slopes have a gradient of 10–20 °. The major part of the slope is formed by flysch sediments of Cretaceous to Palaeogene Age. These sediments are unconformably overlain by Quaternary conglomerate.

A comprehensive engineering-geological survey of the Mingchukur landslide was carried out. This involved a geophysical survey that was followed by long-term monitoring. Field geophysical measurements were made by an Uzbek team and Czech geophysicists participated in the reinterpretation of the original geophysical results. The geoelectrical measurements were used successfully to identify two steps in the landslide. Each step consists of two layers. The resistivity of the lower layers in each step was shown to be quite different. In the top step, the resistivity of the lower layer is almost one order of magnitude higher than that of the lower layer in the bottom step. The top step of the landslide consists of disrupted Quaternary conglomerate and Tertiary flysch. Because they are intensely fractured and have a low moisture content, resistivities are in the range 350–1,500 Ωm. The surface layers have resistivities up to five times higher. The lower layer in the bottom step of the landslide has resistivities in the range 16–30 Ωm. Based on these measurements it can be deduced that the bottom step of the landslide consists mainly of flysch that lies mostly below the groundwater level or that becomes flooded when the water in the reservoir is at its highest level. The increase in moisture, combined with the higher clay content of this step of the landslide, reduces the resistivity of the rock environment. It is possible that the boundary between the first and the second layers in each step corresponds with the groundwater level.

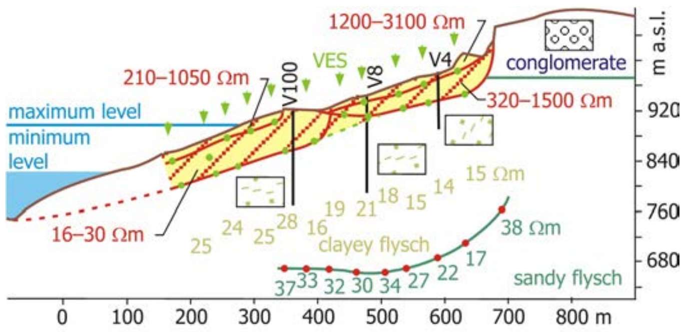


Fig. 6.1.2 Cross-section of the Mingchukur landslide on the bank of the Charvak reservoir

The layer beneath the landslide has a significant thickness and resistivities that vary within narrow limits (14–25 Ωm). The position of the deepest interpreted boundary is interesting. Across it, the resistivity of the rocks increases by about 50 %. The shape of the boundary suggests that it could be formed by the trough limb of a syncline. A change in the dip of bedding of the flysch sediments was also observed in drill core obtained from the site. The change in the dip of bedding is most probably caused by flexure due to folding. This change in the dip of the bedding planes favours sliding, because the plane of failure responsible for the slide does not have to shear across sedimentary layers at a high angle, but can follow those beds in the flysch complex which are weaker or where frictional resistance is reduced.

The construction of every dam requires an adequate supply of construction material. Depending on the type of dam and the ancillary facilities that are to be constructed, there will be differences in the quantity and quality of construction materials that are required. In all cases, it will be necessary to survey the area of the dam for suitable soils and rocks, and delineate the patterns of fracturing and potential workability in the rock mass where quarries could be sited. At a later stage it will be necessary to obtain information about groundwater conditions at sites where excavations will take place so that the stability of the walls can be assessed. The geophysical methods applied at different stages of the survey will depend on the type of information required by the engineering geologist.

The survey for a quarry to provide construction material for the Dlouhá Loučka dam (Synek, 1977, Veselý, 1977) is a good example. At this point it is necessary to explain that the geological structure of this site made the successful application of geophysical measurements and their interpretation extremely difficult. This is because the bedrocks are a flysch facies of the Culm. They show very little variation in their physical properties that could be measured by conventional geophysical methods. Work at many sites has shown that it is almost impossible to distinguish between individual rock types in solid flysch sequences, particularly where there are gradual transitions in sedimentary composition.

At this site, a combination of vertical electrical sounding, resistivity profiling and shallow seismic refraction was used. The results obtained from the profiling methods are shown in Figure 6.1.3, on which a composite section was also plotted on profile P24. Such composite sections were drawn up on all the measured profiles. Despite all the problems that had been predicted, it turned out that the rock mass at the site of the survey was not significantly fractured tectonically. Where it was possible to observe indications of "conductors" in resistivity profiling curves, they were not strongly conductive zones that would be indicative of intense fracturing of the rock mass and clay alteration within the weakened zone. In this case, the rocks had been affected only locally by brittle fracturing and these local zones of weakness could not be correlated over significant distances. These were thus considered as having only a limited effect on the behaviour of the rock mass.

The results obtained from vertical electrical sounding showed that there was a near-surface layer with lower resistivities than the intact rock mass. The reason for this decrease in resistivity is weathering of the rock and formation of clay coatings on the surfaces of the open fractures. Using VES it is possible to determine three main types of lithology (A–C) within the intact rock mass of which rock type B has the highest resistivity. It probably corresponds to greywacke of the Culm sequence. Rock type A is probably the most variable. It incorporates all types of sedimentary rocks found within the Culm, ranging from shale and greywacke shale through siltstones to greywacke, and, possibly also coarse conglomerate. Greywacke shale was intersected in all four exploratory boreholes. The last type, C, has the lowest resistivities and obviously corresponds to shale. In all rock types, however, local changes in the grain size of the sediments can be anticipated, so that there will be some variation in the physical properties of the units distinguished. It seems likely that the layers have a NE–SW strike and a dip to the southeast of about 45 °. The same dip of bedding and lamination in the shale was also observed in drill core recovered during the survey. The uppermost surface layer, which is less than one metre thick, is not depicted on the sections.

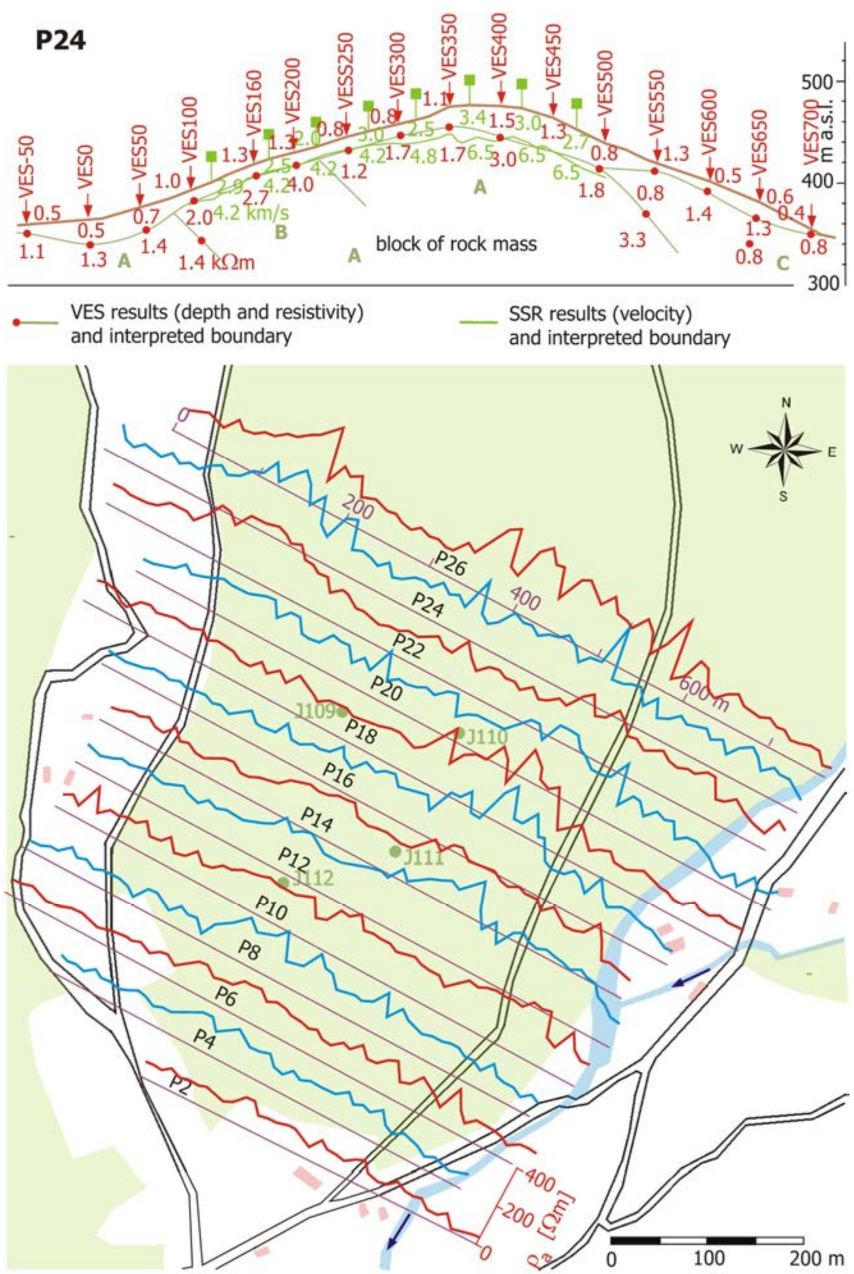


Fig. 6.1.3 Survey for a quarry at the Dlouhá Loučka dam (adapted from Synek, 1977 and Veselý, 1977)

Shallow seismic refraction was interpreted using the Hagedoorn method. As in the case of vertical electrical sounding, the surface layer with velocities less than 0.5 km/s is not plotted in the section. In general, the interpreted velocities are fairly high and velocities above 5.0 km/s are probably unrealistic. It is possible that the shape of the topography increases the absolute values of velocities. Seismic measurements confirm the results of geoelectrical measurements showing that the rock mass is not markedly fractured. Where the seismic boundary fluctuates, particularly in the top part of the profile, this is probably due to differences in the depth of weathering. It is likely that in some places the weathering penetrates along fractures to greater depths than is indicated by the results of the seismic measurements. Seismic measurements show that ripping can be used effectively to a depth of about 10 metres during excavation, and only locally to greater depths. At greater depths, it will be necessary to use explosives to break the rocks, regardless of the composition of the bedrock.

Specific requirements for hydraulic facilities are defined at different stages of an engineering-geological survey. It is therefore necessary to provide the answers to questions about the foundations of the structures at the appropriate time. In this case, the task is similar to that involved in the earlier stages of the survey of the dam site itself.

An illustration of a survey for a hydraulic facility at the Tršice dam is given in Figure 6.1.4. The dam profile is situated in an area where the tip of a Neogene basin lies on Culm rocks. The fill of the Neogene basin consists of claystone, which weathers very readily into clay. The Culm massif is formed by greywacke and shale. The Quaternary cover on the slopes consists of deluvial silty clay and in the valley there are gravel and alluvial silty clay.

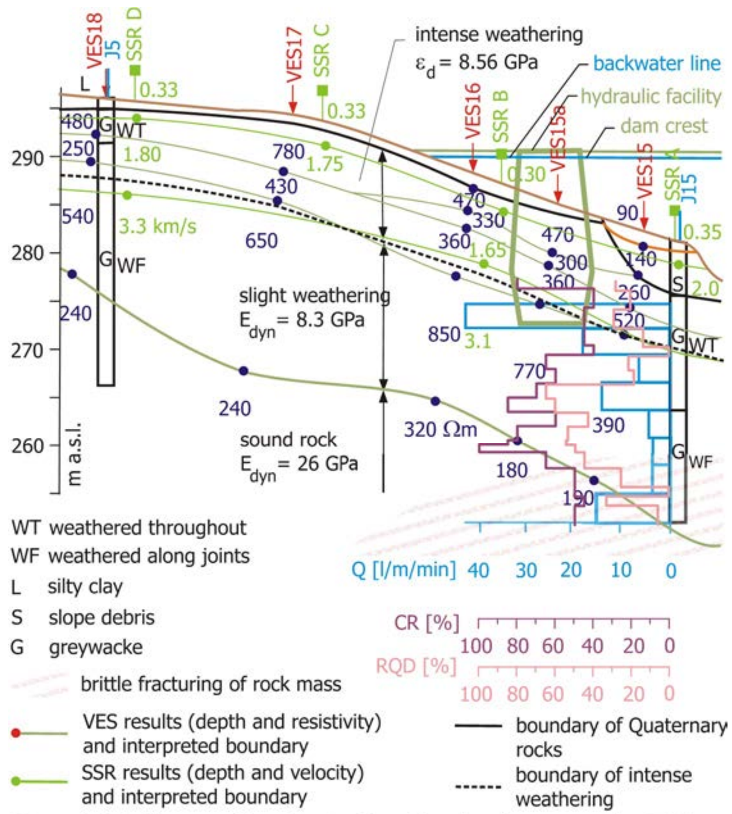


Fig. 6.1.4 Survey for the hydraulic facilities at the Tršice dam site

The survey was carried out by a combination of geoelectrical and seismic sounding methods. Due to the fact that it was a survey for a combined facility, the spacing of points chosen for VES was short. At the site of the designed facility and in the immediate vicinity, the spacing used was ten metres, and at greater distances the spacing used was approximately twenty metres. In the vicinity of the hydraulic facility, the geophysical survey successfully distinguished between Quaternary soils and Culm rocks, the degree of weathering of which could also be successfully determined. In the zone of intense weathering, it was even possible to distinguish several sub-layers based on the geoelectrical measurements. It is presumed that the differences in these sub-layers are due to variations in the extent to which fractures are filled by the clayey weathering products.

Based on the results of this geophysical work, a pattern of boreholes was proposed for a higher stage of the survey. It is shown in the figure so that it can be seen how the interpretation of the geophysical measurements agreed with the results of the drilling. The main lithological types, the degree of weathering, the results of water pressure tests and the core recovery, including RQD, obtained as a result of drilling work are illustrated. The results of water pressure tests and the decrease in core recovery indicate that the rock mass at the base of borehole J15 is fractured. Comparison of these observations with the results of geophysical measurements shows that the fracturing of the Culm rock mass is not accompanied by degradation. The fracturing of the greywacke complex can be described generally as brittle. The detailed description of the drill core shows that calcite-filled veinlets were documented at this depth, suggesting that brittle fractures in the rock mass were subsequently healed by secondary carbonate minerals. Based on the experience gained from the survey of the Culm rocks, it could be concluded that this fracturing does not affect the whole of the bedrock at the site, but is confined to particular zones. The shape of the geophysical boundaries shows that the trend of the fractured zone is oblique, both in the plane of the profile and also perpendicular to the profile. The use of shallow seismic refraction at the preliminary stages of this survey had already enabled the boundary of intense weathering and the basic geotechnical parameters of the rocks to be defined. The hydraulic facility was sited successfully where the Quaternary cover was thinnest and, at the same time, in an area where it could be predicted that excavation could be carried out without using explosives to break the bedrock.

In the process of dam construction, as in the case of other large civil engineering projects, at some points it becomes necessary to carry out checks and test measurements for a variety of reasons. Geophysics offers a range of possibilities for making such measurements. Sometimes additional surveys are required because of unusual problems identified at an earlier stage of survey work.

The following example is an illustration of measurements used to check a landslide at the Ujala site. When interpreting logging measurements made in an inclinometer borehole lined with grooved plastic casings, a quite unusual pattern of the curves in the gamma gamma log (GGL) and velocity log (SL) was detected in the interval over the depth from 11 to 14 metres. For this reason, a log of magnetic susceptibility was subsequently carried out. In Figure 6.1.5, only selected curves from the whole range of logging measurements are plotted and only the most interesting part of the log of the borehole down to a depth of 30 metres is shown.

The increase in the velocities of longitudinal waves over the interval from 11 to 14 metres, together with the sharp increase in the bulk density (GGL curve) could not be explained by any obvious geological causes. Only the log of the magnetic susceptibility measurements (MSL) offered a clue. In the section shown, a sharp increase of susceptibility was detected, large enough to be outside the range that the probe could measure. The only possible cause of such an anomaly would be the presence of ferromagnetic material behind the plastic borehole lining; in this case it was a steel casing. The presence of steel in the space between the plastic casing and the rock mass is indicated by the anomalies in the GGL and MSL curves. The presence of a steel tube at such a depth in the borehole could only be explained by a faulty procedure when the hole was being fitted out.

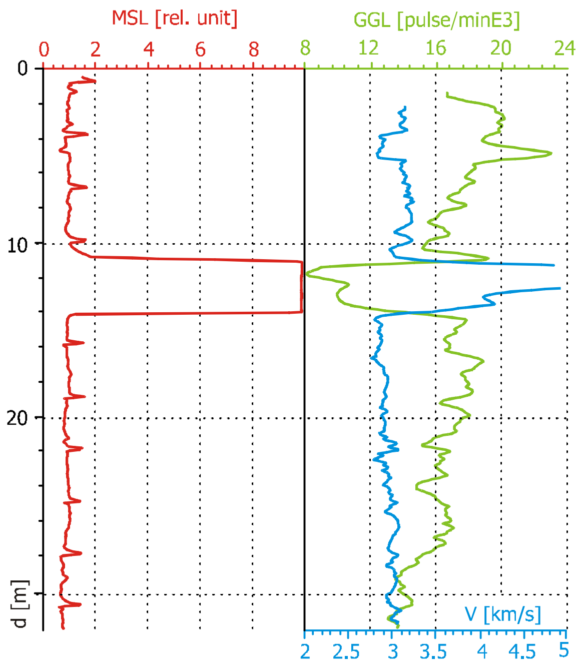


Fig. 6.1.5 Geophysical logs used to check a borehole at Ujala (adapted from Lukeš, 2003)

Evidently, at the final stage of completion of the inclinometer borehole, a mistake was made so that a 3-metre length of protective steel casing dropped down to a depth of 11 to 14 metres. It should be noted that no information about the fallen casing was given in the technical report by the drilling company. This explanation was confirmed by the fact that there was no indication in the MSL curve of a standard length of protective casing at the surface. The protective tube had originally been embedded in concrete only about 15 cm below the surface. This fact was also confirmed directly some time later. The protective steel casing was stolen by metal thieves. An inspection carried out using a television camera showed that the steel casing had reached a depth of only 0.3 metre below ground. Small anomalies on the MSL curve, which are repeated at three-metre intervals, are an indication of riveted joints in the inclinometer casings. Fallen steel casings can also be identified by measuring the natural high-frequency electromagnetic field along the axis of a borehole.

Geophysical measurements carried out during the operation of a dam facility are mostly designed to answer questions about the safety of operations or to test the possibility of increasing the capacity of a dam. An example from a survey of the Dolní Líštná site is given in Figure 6.1.6. In this case the aim of the survey was to test the feasibility of raising the dam at the sludge lagoon of Třinec Ironworks by 20 metres. Due to safety considerations, the use of direct survey methods was limited, and therefore a survey was carried out by a combination of geophysical methods using vertical electrical sounding, shallow seismic refraction and symmetrical resistivity profiling. Because questions about the potential seepage of water through the body of the dam had been raised, the method of spontaneous polarization (SP) was also used.

Geophysical profiles were sited along the dam crest and on the downstream side of the dam body, both in longitudinal and perpendicular directions. The measured profiles were spaced about 20 metres apart, and the spacing between the points at which VES and SSR measurements were made varied from five to ten metres. The spacing between measurements along the profile was 2.5 metres.

On the right of Figure 6.1.6, the results of the measurements of spontaneous polarization are shown. The SP measurements on individual profiles were interconnected and the value of the normal field was determined statistically. At three points, positive SP anomalies were detected. The relative size of positive anomalies ranged up to 28 mV. These were identified as the places at which seepage of water from the reservoir was occurring and warnings were therefore given about the possibility of water bursting out. This prediction was subsequently confirmed when groundwater irruption occurred in area "A" a month after the geophysical survey was completed.

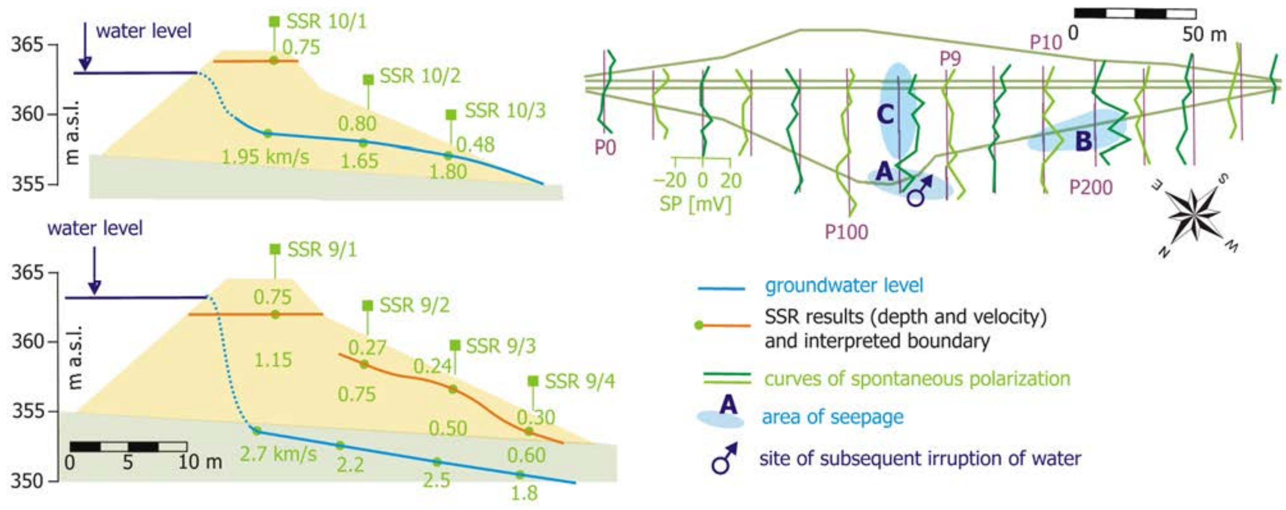


Fig. 6.1.6 Dolní Líštná - detection of seepage through a dam body

The differences in the elevation of the groundwater level in the dam body were also indicated by the results of shallow seismic refraction. In the left part of the figure, two profiles with different results obtained from SSR are illustrated. The interpretation of these two profiles depends on the fact that the velocity of propagation of longitudinal waves rises significantly in soils as the moisture content increases. In the lower section, the groundwater level (GWL) corresponds to the theoretical elevation of the GWL when the sealing system of the dam is working well. In the upper section, the seismic boundary marking the GWL lies at greater depths indicating that groundwater is escaping through the body of the dam.

6.2 Choice of Geophysical Methods

There is a very wide range of sounding and profiling methods that can be used to carry out geophysical surveys and solve engineering-geological problems associated with particular dam sites. The methods chosen and the way in which they are combined depend on the requirements imposed by the designer of the dam and its associated facilities, as well as on the geology of the site itself. The physical properties of the rocks and soils, and the fracturing of the rock mass will always be important factors to consider. Despite these constraints, certain general principles can be applied to the selection of individual geophysical methods. The uses of individual geophysical methods for the solution of particular problems at different stages of an engineering-geological survey are listed in Table 6.2.1.

It is obvious that the scale of any survey and the geophysical methods that can be used will depend on the amount of money allocated from the budget for this purpose. However, it must be borne in mind that, in many cases, geophysics is the only way that certain information can be obtained. It is also true that a well-conducted engineering-geo-logical survey, of which geophysics forms an integral part, will lead to significant savings in the overall cost of a project because of increases in technical efficiency and safety.

Table 6.2.1: Choice of geophysical methods

	Geophysical method	Task						Stage			
		a	b	c	d	e	f	o	p	d	a
Geoelectrical	Vertical electrical sounding	●	●	●	●	✦	✦	●	●	●	●
	Resistivity profiling	●	●	●	●	✦	✦	●	●	●	●
	Electrical resistivity tomography	●	●	●	●	✦	✦	✦	●	●	✦
	Electromagnetic profiling	●	●	●	✦	✦	✦	●	●	●	✦
	Georadar	●	●	✦	✦	✦	✦	✦	●	●	✦
	Electromagnetic sounding and transitional phenomena	✦	✦	✕	✕	✕	✕	✦	✦	✦	✕
	Spontaneous polarization	✦	✦	✦	✦	✦	✦	✦	✦	✦	✦
	Induced polarization	✦	✦	✦	✦	✕	✕	✦	✦	✦	✕
	Mise-a-la-masse method	✦	✦	✦	✦	✕	✕	✦	✦	✦	✕
	Metal detectors	✦	✦	✦	✦	✦	✦	✦	✦	✦	✦
	Very long frequency method	●	●	●	✦	✦	✦	●	●	●	✦

The strategy for a geophysical survey of a dam site will be determined by the geophysical methods that are chosen, the time and money available, the procedures used and the spacing of the profiles and the intervals between points at which the geophysical measurements will be made. The nature of the site in question and the questions to be answered will be crucial factors governing the choice of methods used and the spacing of measurements. In general, it is true that more information will be acquired if several geophysical techniques are combined and the spacing between points at which measurements are made is smaller. Obviously, the more detailed the procedure, the greater the cost. Because of this, it will always be necessary to balance the interests of the client against the technical demands of the project so that the geophysical survey can be carried out efficiently to provide the maximum information at the lowest cost. It is the duty of the engineering geologist and the applied geophysicist to reach a reasonable compromise by persuading the client that money saved by

Seismic	Shallow seismic refraction	●	●	●	●	●	●	●	●	●	●
	Shallow seismic reflection	●	●	✦	✦	✦	✦	●	●	●	✦
	Seismic tomography	●	✦	✦	●	●	●	✦	●	●	●
	Microseismic zoning	✦	✕	✕	✕	✕	✕	✕	✦	✕	✕
	Acoustic measurements	●	✦	●	●	●	✦	●	●	●	●
Logging	Logging measurements for characterization of lithology	●	✦	●	●	✦	✦	●	●	●	●
	Logging measurements for characterization of mechanical properties	●	●	●	●	●	●	●	●	●	●
	Hydrogeological logging	●	✦	✦	✦	✦	✦	●	●	●	✦
	Inclinometry	✦	●	✦	✦	●	✦	✦	●	●	●
	Optical logging	●	●	✦	●	✦	✦	●	●	●	●
Mag.	Profile measurements	●	●	●	✦	✦	✦	●	●	●	●
	Measurements of magnetic susceptibility	●	✦	✦	●	✦	✦	✦	●	●	✦
Grav.	Profile measurements	✦	✕	✦	✦	✦	✦	✦	✦	✦	✦
	Measurements of bulk density	✦	✦	✦	✦	✦	✦	✦	✦	✦	✦
	Thermal measurements	✦	✦	✦	✦	✦	✦	✦	✦	✦	✦
	Radiometric measurements	✦	✦	✦	✦	✦	✦	✦	✦	✦	✦
	Remote sensing and aerial geophysics	●	●	✦	✕	✦	✦	●	●	✦	✦

Explanatory notes:

a	*Survey of dam side*	*o*	*Orientation survey*	●	*Suitable for use*
b	*Survey of future reservoir*	*p*	*Preliminary survey*	✦	*Possible to use*
c	*Survey for construction materials*	*d*	*Detailed survey*	✕	*Unsuitable to use*
d	*Survey at sides of future structures*	*a*	*Additional survey*		
e	*Measurements made as a check during survey and construction*				
f	*Work during operation of facility*				

limiting the scope of a geophysical survey at an early stage can result in much greater losses at later stages if there are technical failures during construction work, or accidents occur because information about the geological risks involved is not available.

Figure 6.2.1 illustrates the fact that even a single geophysical method can yield useful and practically acceptable results. In this case, seismic refraction was used to carry out a survey for the portals of access and cable tunnels. The problem was to determine the depth at which the bedrock lies (Velen *et al.*, 2008). The measurement of three profiles enabled the depth of the bedrock to be determined as 40 to 70 metres. The depth range of the measurements meant that blasting was used as a source for the seismic signal. Based on the results of these measurements, the decision was taken to move the portal of the access tunnel towards the cable tunnel and to construct a single portal for both tunnels. Based on these seismic measurements, it was also possible to make a decision about the position of other investigation works. In addition, the contours of velocity provide clear evidence that there is a zone of fractured rock at the beginning of each profile. It is impossible to determine from the measurements that were made whether this is a single zone of variable width or two thinner zones which intersect at an acute angle.

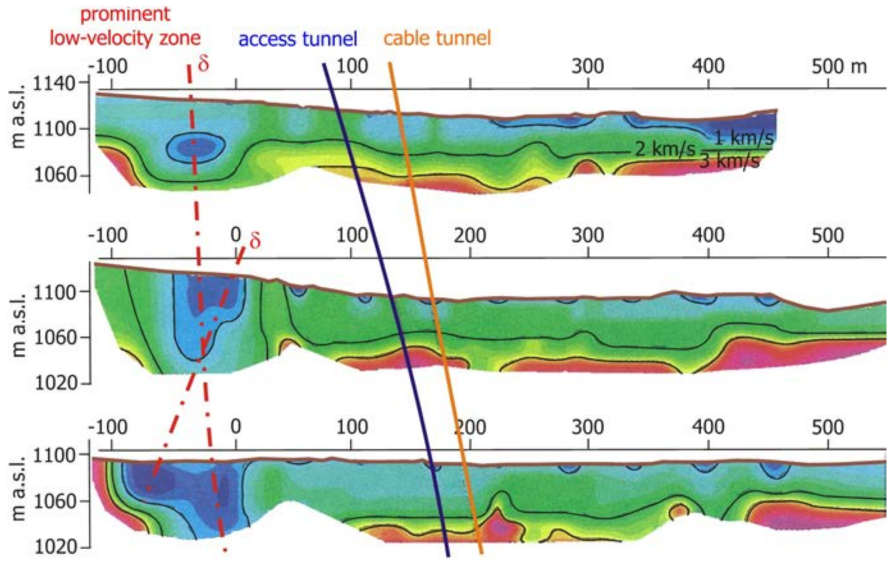


Figure 6.2.1 Seismic refraction survey along profiles (after Velen et al., 2008)

Another example is the use of seismic methods for checking the level of compaction of an earth-fill dam (Fig. 6.2.2). A dam eighty metres high on the River Rezaksay was constructed using local gravel as a construction material. It was checked during construction applying standard procedures and also by using geophysical methods. A correlation was established between the bulk density of material determined by routine procedures and the values obtained from parallel seismic measurements (Abdullaev, 2008). It was shown that the correlation was sufficiently close to safely reduce the number of routine tests. The application of the shallow seismic refraction thus permitted the time necessary for routine tests to be shortened. In this case, the effective use of geophysics led to a faster rate of construction without sacrificing the quality and safety of the dam.

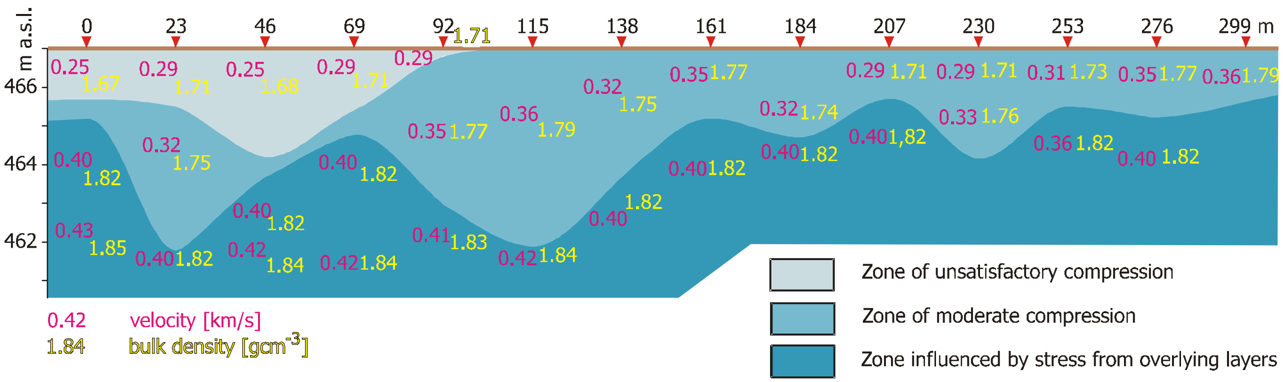


Figure 6.2.2 Shalow seismic refraction of a monitoring profile on the earth-fill dam on the River Rezaksay showing the effects of compression (adapted from Abdullaev, 2008)

6.3 Use of Geophysical Methods

The assumption is made that the readers of this book understand the basic rules governing a geological survey, and the principles used in complementary geophysical surveys. For this reason, the basic techniques used to make geophysical measurements and interpret the results will not be described in any detail below. The aim of this book is to focus on examples in which the use of common geophysical methods in engineering-geological surveys differs from the way in which they are used for the purposes of a conventional geological survey and particular attention will be given to those geophysical techniques which are of specific use in engineering-geological, hydrogeological and geotechnical surveys for dams.

In the Czech Republic, and in the former Czechoslovakia, geophysical methods began to be used for engineering-geological purposes at the end of the 1960s and in the early 1970s. Before describing and assessing the value of the individual methods used in geophysical surveys, some comments about changes in geophysical techniques and the interpretation of results are appropriate. Certainly, improvements in procedures for handling, interpreting and presenting large sets of data using computers have taken place, but the most important advances have been achieved through the close cooperation between specialists from all the branches involved, notably engineering geologists, geotechnical specialists and geophysicists.

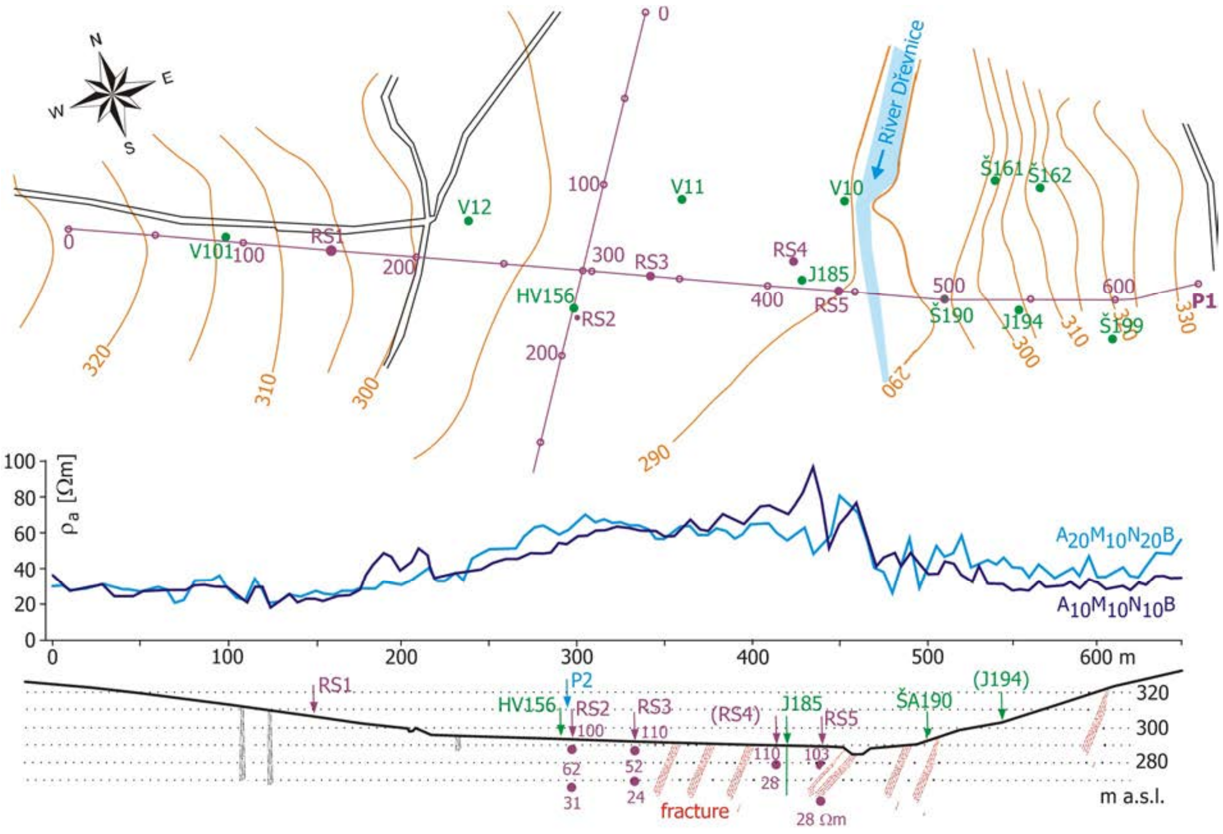


Fig. 6.3.1 Early geophysical survey of the Slušovice dam site

Figure 6.3.1 shows the results of a geophysical survey carried out at the Slušovice dam site. This is one of the first dam sites in the Czech Republic where geophysical methods were used as a part of engineering-geological investigation. Thanks to modern graphic techniques, this illustration is not simply a copy of archived material. The map of the area and the surveyed profile has been redrafted in colour. Apart from this, no changes have been made to the original interpretation.

The area of interest is formed by rocks of the Magura Flysch, which are folded and over-thrust onto the Outer Flysch. The geological structure is determined by anticline structures with longitudinal axes, which are truncated by thrust faults in the north and north-west of the area under investigation. The area is underlain by alternating sandstone and claystone of the Zlín Beds. Based on the results of direct exploratory workings, the rocks are intensely fractured as a result of tectonic deformation. The strike and dip of beds does not remain constant along the dam profile. Like the figure above, a part of the original text from the early 1970s describing the survey is reproduced here:

"The task of the preliminary geophysical survey was to delineate zones of pronounced weakness in the dam profile and to try to answer questions about the tectonic factors, if any, that governed the development of tributary valleys on the right bank. Because of the lack of parametric measurements in the area of interest, measurements were made on certain boreholes using vertical electrical sounding during the first stage of the fieldwork. This was done in order to determine the resistivities of individual rocks and to define the pattern of the resistance field in the vertical direction. Based on this preliminary survey, resistivity measurement was chosen as the preferred geophysical method. For resistivity profiling, the basic spacing A10M10N10B was selected, which was complemented by the spacing A20M10N20B. It is possible to determine the apparent dip of weakened zones from the results of measurement using both these spacings.

It is possible to identify weakened zones at the site of the designed dam using the results of symmetrical resistivity profiling shown in profile P1. Resistivity minima were used to identify and correlate the weakened zones. These can be affected by tectonic fracturing, or by an increase of the clayey component in individual layers, especially in the places where sandy sediments prevail. The most obvious zone of weakness in the entire profile is an interval from 350 to 540 metres, where the pattern of apparent resistivity obtained using both the spacings is very jagged. This indicates that this section has been subject to intense tectonic fracturing. The most pronounced zone of weakness was encountered at 450 metres at point VES5. In the western section of the profile (to 240 m), clayey layers prevail. The intermediate zone reaches to 470 metres and consists of a predominantly sandy sequence. Farther on, up to the end of the profile, clayey layers again prevail."

Another more recent example dates from the time when more extensive experience of using geophysical methods for engineering-geological surveys had already been gained. In this case it was possible to carry out a survey of a much wider area around the site of interest and also to use a more closely spaced pattern of measurements. A more confident description of the geological structure of the area could thus be given, and the characterization of individual units of the investigated rock mass was much more precise.

The example described here is a survey of the valley of the Syrovátka brook in the vicinity of Křenovice (Fig. 6.3.2). In this case, the surface layer having higher resistivities is interpreted as loess. In the floodplain, a near-surface layer with lower resistivities than loess is also present. This layer can be interpreted as a layer of flood silty clay. Valley gravel is probably absent. If it was present, it would contain a high proportion of silty clay, and solid rock fragments would not form the supporting matrix of the sediment. On the right slope, a layer is depicted which is probably a stabilized slope failure (layer E). From the pattern of apparent resistivities shown by the SRP it is likely that the slope failure is not active and only the shape of the boundary defining the base of the layer with that value of resistance indicates that a slope failure exists.

Between the Quaternary and Tertiary sediments, there is a layer F (on the left slope) and a layer K (on the right slope). Based on the borehole logs, both of these layers can consist of the eluvium derived from the original sediments and the upper *in situ* layer of the Neogene. It is very difficult to make a more detailed geotechnical characterization of these layers based on geoelectrical measurements. It can be deduced only that the sediments are very clayey and that decreases in resistivity are due to higher moisture contents than are present in the relatively unweathered Neogene sediments.

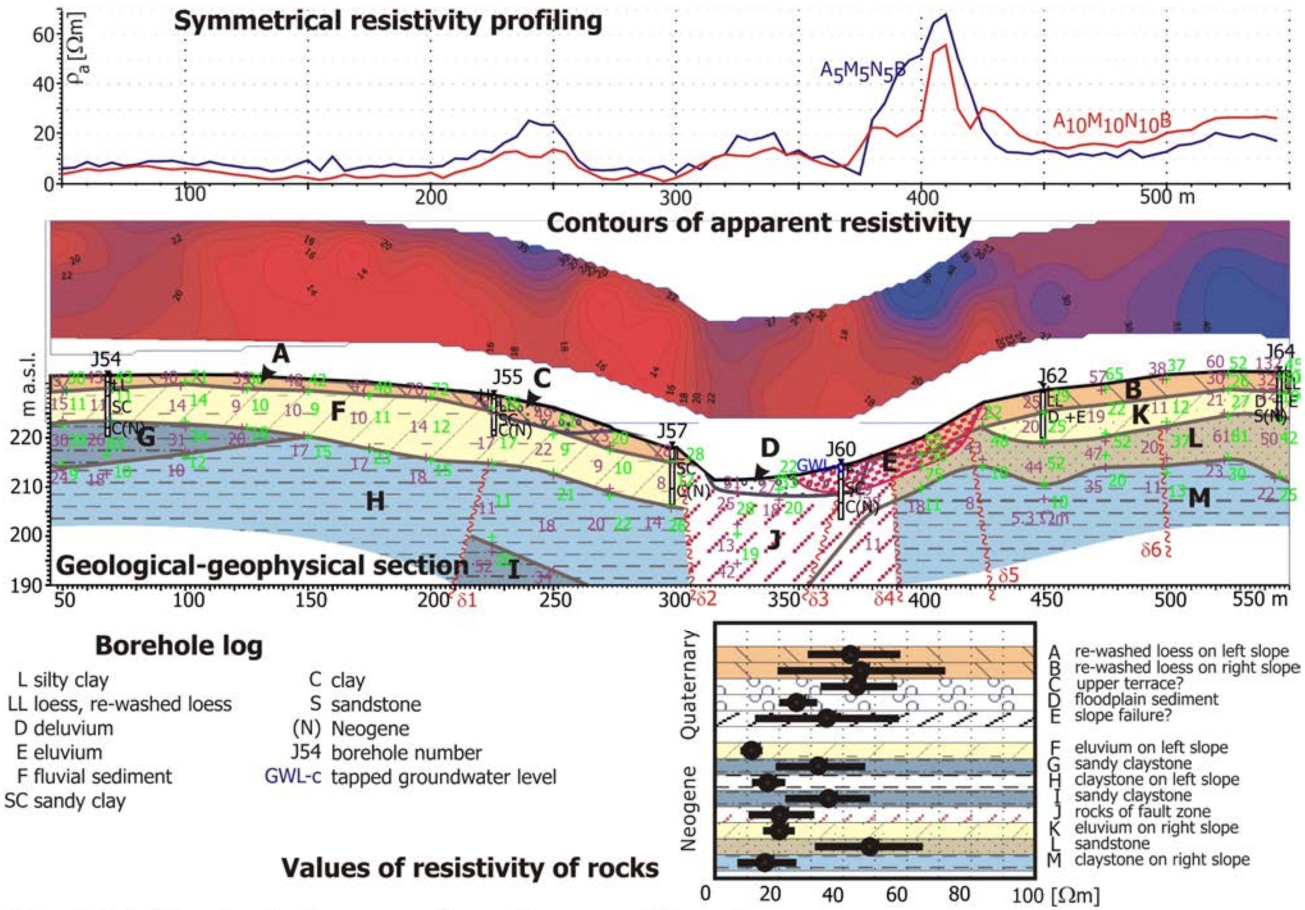


Fig. 6.3.2 Geophysical survey of a valley near Křenovice

The Neogene sediments on the left slope are formed by layers G, H, and I. Layer H corresponds to the Neogene claystone, layers G and I to sandy claystone. The Neogene beneath the valley bottom consists of a discrete unit, which is denoted as layer J. Based on the geoelectrical measurements, it is not possible to determine its lithological composition because this belt of rocks is tectonically fractured and the fracturing prevents a conclusive identification of the original lithology based on the interpretation of the measured resistivities of the medium. The Neogene sediments on the right slope consist of two different units, layers L and M. According to the borehole log of J64 and the values of resistivity, layer L was interpreted as sandstone, and layer M as claystone.

To enable comparison of the properties of the separate layers identified on the basis of measurements of resistivity, the average value and the range of resistivity for each layer is calculated. Based on this procedure, the physical similarities between individual units can be compared so that the lithological properties and the extent of fracturing can be interpreted. The eluvia on both the left and the right slopes have lower resistivities than those measured in the underlying layers. There are a number of possible reasons for this. One is that the rocks are strongly weathered, another is that loamy and clayey components from the surface layers have been washed into the open spaces in the eluvium and also the eluvium contains more water than the underlying rocks. Based on the values of resistivity the claystone on the left slope has physical properties similar to those on the right slope and both belong to the same stratigraphic unit.

By processing the results of the resistivity survey in the way described above, it was possible to provide a credible interpretation of the stratigraphy and geological structure of the Křenovice site. Based on the results of this preliminary survey, it became evident that a more detailed investigation of the immediate vicinity of the brook should be carried out at a more advanced stage of the survey. The reasons for this were the tectonic fracturing of the rock mass beneath the valley bottom and on the adjacent slopes, and the existence of an old slope failure on the left bank of the brook detected at the early stage of the survey.

Another step in the introduction of geophysical methods to engineering-geological practice in the mid-1970s was the transition from measurements along individual profiles to surveys over defined areas. The application of geophysical methods to the survey of definite areas is an advantage in poorly explored sites, in particular where there are a number of options for the location of a dam and the associated facilities and infrastructure. An example of such a survey is that carried out at the Hrhov site. This survey was the first areal application of geophysical methods to engineering geology in the former Czechoslovakia (Fig. 6.3.3.).

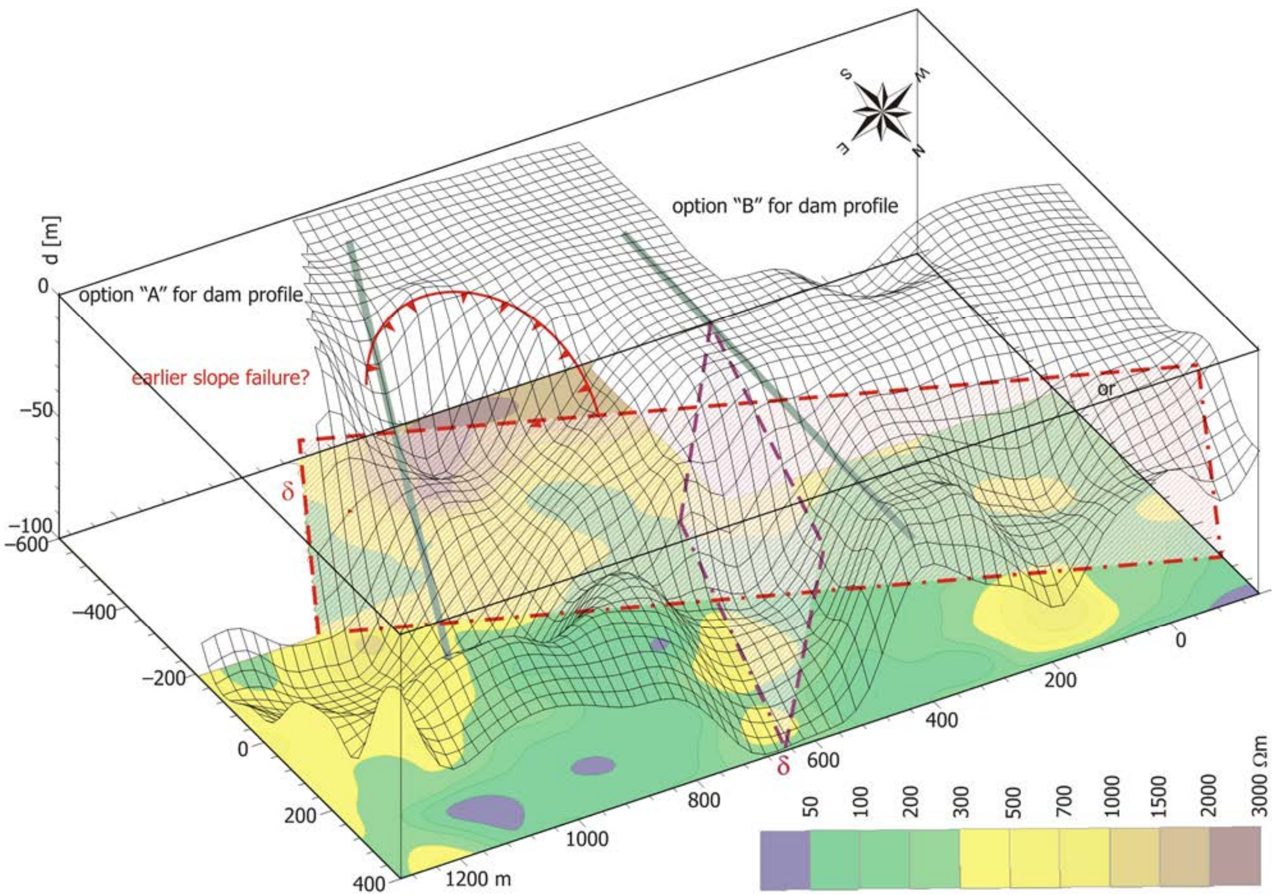


Fig. 6.3.3 Survey of the area for siting the Hrhov dam and power plant

In this case, the survey was not made for a conventional valley reservoir, but for a pumped storage hydroelectric plant, involving the location of a suitable site for a lower dam and a place to build a surface powerhouse. The area lies within the South-Slovakian Karst. In the area of the lower reservoir, the future powerhouse and the lower part of the penstocks, the area is underlain by Mesozoic rocks, namely the flyschoid rocks of Werfenian and Campilian age. The Werfenian rocks consist predominantly of claystone with beds of siliceous sandstone. The Campilian rocks have a more calcareous character, but the clayey component present in the rocks prevents extensive development of karstic processes. The upper structural level is formed by rocks of Neogene and Quaternary age. These sediments occur only on the bottom of the Turnia Basin where they reach a thickness in excess of 100 metres. Based on the results of drilling, it was known that the character of the Neogene sediments changes abruptly and that they largely consist of clay and sandy clay. At the edges of the Turnia Basin, the Neogene sediments contain a large amount of admixed clastic material carried down from the adjacent slopes. These sediments can be cemented by carbonates.

The areal survey carried out by vertical electrical sounding combined with resistivity profiling covered a rectangle of 1,300 × 600 metres within which it was possible to locate the surface powerhouse and the left flank of the dam. Another area of about 300 × 650 metres lies where the right flank of the dam would be sited. The results of the interpretation of VES over this area are given in the block diagram The "wire" frame depicts the surface of the Mesozoic sediments and the coloured areas mark the variations in the resistivity of the Mesozoic sediments.

The most significant feature of the geological structure of the Neogene basement is a fault trending in a WNW–ESE direction, which dips towards the NNE. Along this fault, the north-eastern block has been thrown down to depths of over 100 metres. Another dislocation runs oblique to this fault in the direction N–S. The dip of this structure cannot be determined from the geophysical measurements. An interesting anomaly was identified in the basement in the area of co-ordinates 900/-200. The form of this anomaly is strongly suggestive of a slope failure. This must have affected the slopes formed by Triassic sediments before the beginning of sedimentation in the Neogene.

The areal distribution of resistivities on the surface of the Werfenian and Campilian sediments shows no simple relationship. It can be observed, however, that the rocks of the sunken block have lower resistivities than the rocks which form part of Horní vrch. The explanation for this may be the more intense fracturing of these rocks, or that the open spaces (cracks and possible cavities) in the limestone intercalations have been filled by clayey material derived from the Neogene sediments. Using the results of the geophysical survey, it was not possible to determine whether these tectonic effects were the result of younger orogenic activity or not.

6.3.1 Geoelectrical Methods

Geoelectrical methods constitute the most extensive range of techniques used in geophysical surveys. One reason is that the spectrum of properties of rock material that influence the shape and size of geoelectrical fields is relatively broad. Resistivity, dielectric constant, permittivity, permeability and polarizability are properties that offer a diversity of options for measuring geoelectrical fields. Another reason is the range over which such properties can vary, for example the resistivity of different rocks and minerals can range over 15 orders of magnitude. Last, but not least, the origin of fields used and their frequencies must also be borne in mind. Nowadays, fields ranging from zero up to gigahertz frequencies are applied for the purpose of geoelectrical surveys.

The first of the examples of the effective use of geoelectrical methods is a case in which a proposed dam profile was eliminated from further consideration by carrying out a relatively cheap survey using vertical electrical sounding. At the Plachtinský brook, a number of sites were identified as potentially suitable for the construction of a new dam. To enable a clearer view of the situation, Figure 6.3.4 depicts only the results of measurement on the right bank in the dam profile.

The surveyed dam profile lies at the boundary between the Krupina Highland and the Ipel' Basin. On the boundary between these two geomorphological units, the topography is dominated by steep slopes formed by Tortonian volcanic rocks, flanked by fallen blocks of large dimensions and slope debris. The area is dissected by cross faults along which individual blocks have been signifycantly upthrown or downthrown. The crystalline or Mesozoic basement is overlain by a thick, subhorizontal sequence of Neogene sediments. These consist of clay and sand with frequent intercalations of pyroclastic rocks. The Quaternary alluvium and dejection cones reach a thickness of up to seven metres and contain abundant fragments of solid igneous rock which are locally the only detrital component in the Quaternary alluvium. On the slopes, the talus can be up to two metres thick.

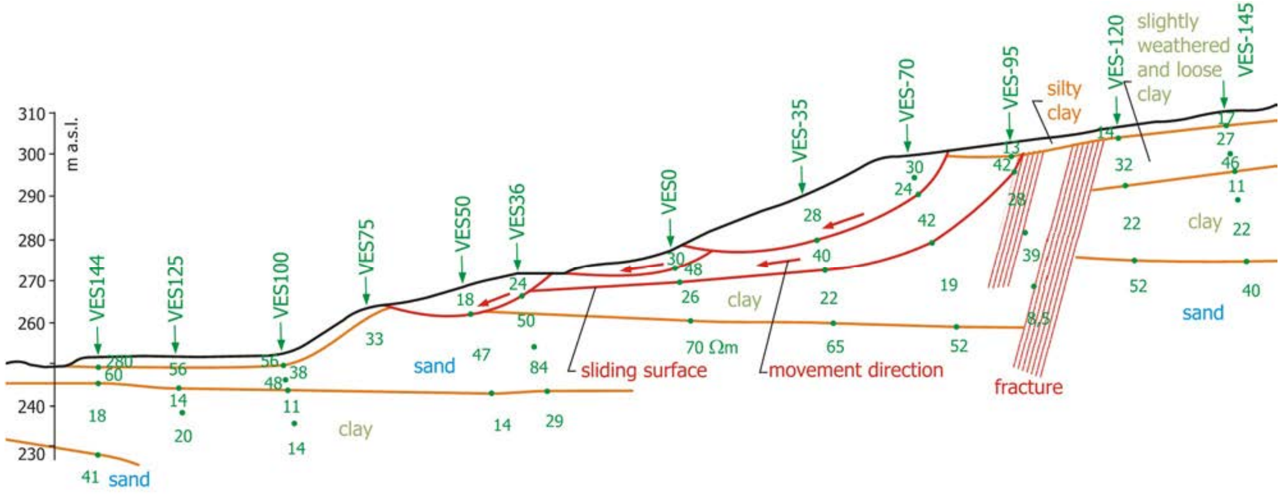


Fig. 6.3.4 Landslides on the slope of the Plachtince dam profile

Analysis of the geoelectrical measurements in the surveyed profile revealed the existence of stabilized slope failures of different ages. Certain sliding surfaces have a typical rotary shape, others are more planar. The susceptibility of the clay to sliding is indicated by their weathering and relief (VES-120 and VES-145). Similar features were identified on the slopes along other profiles in the surveyed area. It is interesting that slope failures have affected not only the clay strata, but also reach sandy layers. These findings enabled the geological structure of the surveyed area to be visualized and re-interpreted. The discovery of the slope failures and the general instability of the area led to the decision to abandon plans to construct a dam at this site. The cost of carrying through the proposed dam would have been financially unacceptable because of the remedial measures that would have been required.

In recent years, new geoelectrical methods have been progressively introduced for geophysical surveys of dam sites. These enable the characteristics of the rock mass and especially its geological structure to be interpreted with greater confidence. The development of these improved methods is chiefly due to innovations in computer technology and related techniques of measurement employing sophisticated apparatus with microprocessors. Major advances have been made in the use of ground-penetrating radar (GPR) and in electrical resistivity tomography (ERT) for surveys of the subsurface. A variety of names have been given to the various procedures employing these two geoelectrical methods, driven by commercial interests that promote their use for survey purposes. Unfortunately, the claims made for the effectiveness of these methods are often exaggerated. It is not unusual for a client to request the use of a particular geophysical method, without being fully aware of the capabilities of the method or the practical and financial implications. It is far better for the client to discuss the problems which need to be solved with the engineering geologist, and leave the geophysicist to select the most appropriate method.

The first example of the use of geoelectrical survey methods is an application of electrical resistivity tomography (Fig. 6.3.5). Synonyms used for this method are: “multi-electrode profiling”, “multi-cable”, “electrical tomography”, and “resistivity tomography”. The upper section of the figure is an illustration of the definition of a contact between Cretaceous sediments and crystalline rocks (Bárta, 2008). The contact between these two blocks of rock is interpreted to lie at 310 metres on the profile and the pattern of distribution of resistivity shows that the contact is perpendicular or steep. The high resistivities indicated by the blue colour near the ground surface in the middle and right-hand part of the profile correspond with a layer of dry debris.

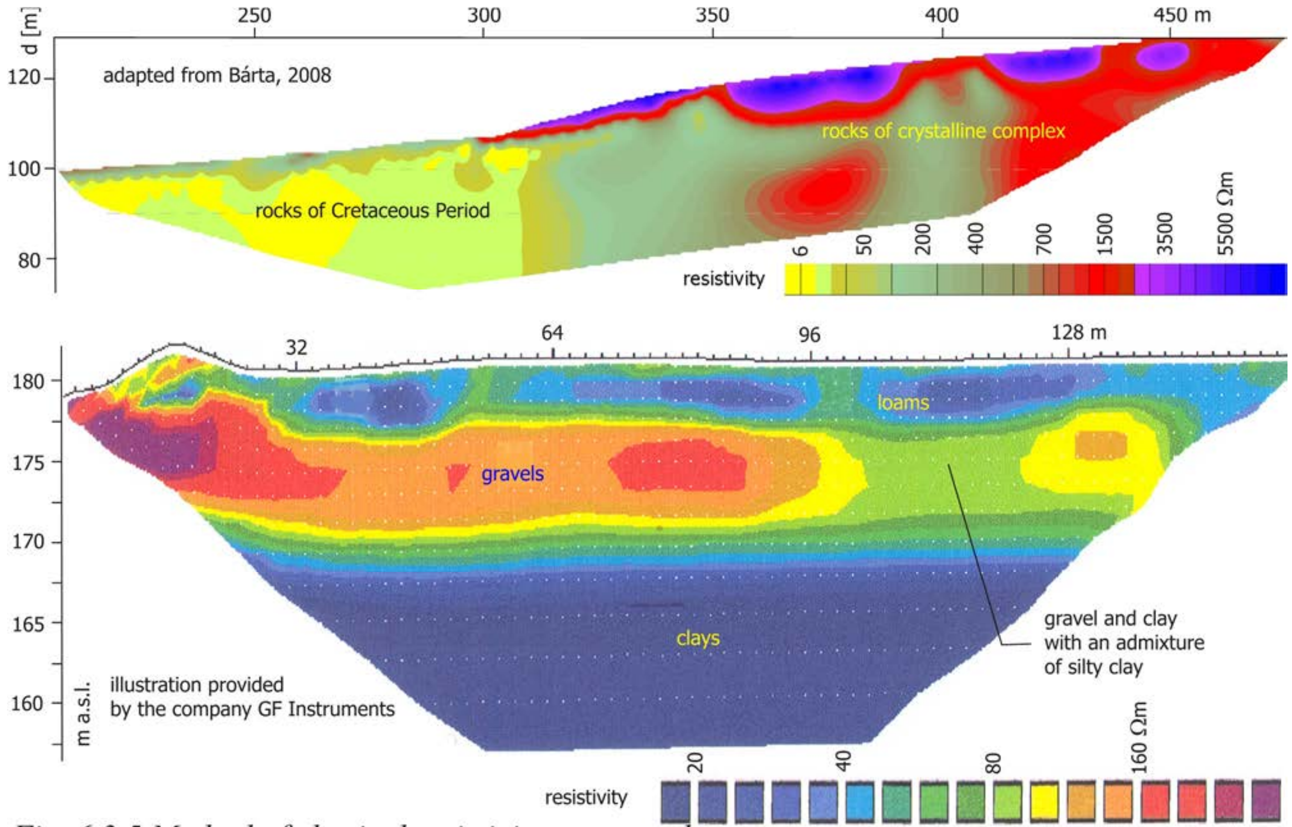


Fig. 6.3.5 Method of eletrical resistivity tomography

In the lower profile, the processed results of another multi-electrode survey are shown. The survey was carried out by the Czech company, GF Instruments, making measurements using an electrode spacing of two metres. The results illustrate the effectiveness of this method for determining the changes in geology that occur at depths of up to 40 m along the surveyed profile in which silty clay, gravel and clay are distinctly identifiable. The only limiting factor in the interpretation of the geology using this method is the agreement between theoretical predictions and the real geophysical behaviour of the rocks in the surveyed profile. If the geological structure is complicated, the results obtained may not correspond fully with the real geological situation.

The results obtained by using “OhmMapper” developed by the American company Geometrics are shown in Figure 6.3.6. They are graphically similar to those produced using multi-electrode profiling. The power used to induce the current is much lower however, so this method is suitable only for surveys down to shallow depths. The method is therefore suitable for surveys of the smaller facilities on a dam project rather than for the investigation of the dam foundation itself. It is particularly useful in areas where the climate is arid. In the original materials of the American company, the same profile is also measured in the opposite direction, i.e. from higher to lower chainage. The results of measurement were not completely identical, but the position of the main anomalies was the same and the interpretation was similar.

Devices used for radar surveys performance can be divided into two groups. The first is designed for geological purposes and uses frequencies from 12.5 MHz to 400 MHz. The radar used for civil engineering work makes use of higher frequencies in the range from 200 MHz to 2.2 GHz. In recent years, this traditional division has progressively disappeared as new devices equipped with supporting antennas at all available frequencies could be employed.

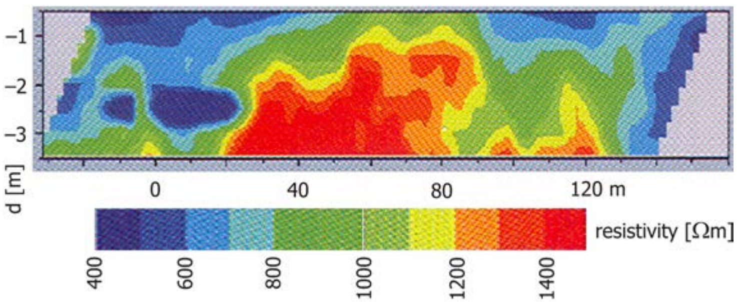


Fig. 6.3.6 "OhmMapper" method (illustration provided by the company Geometrics)

An illustration of the application of ground-penetrating radar to determine the near-surface geological structure is taken from a case study by the Czech company GIMPULS Praha

(Fig. 6.3.7). When using the radar method, high frequencies enable higher resolution, but the depth of penetration is diminished. Radar produces good results when the depth of the survey lies within a few metres of the surface, but in some cases greater penetration can be achieved. In the case of the survey carried out at Teruel in Spain by GIMPULS Praha (Hrubec, 1999), measurements using radar were made along a profile some 300 m long, the results of which are shown in Figure 6.3.7. The field trip was made across a sequence of dolomite and into clay. The contact between these two types of rock occurs at 455 m along the profile. There is a profound difference between the response of these two rock types to the electromagnetic waves.

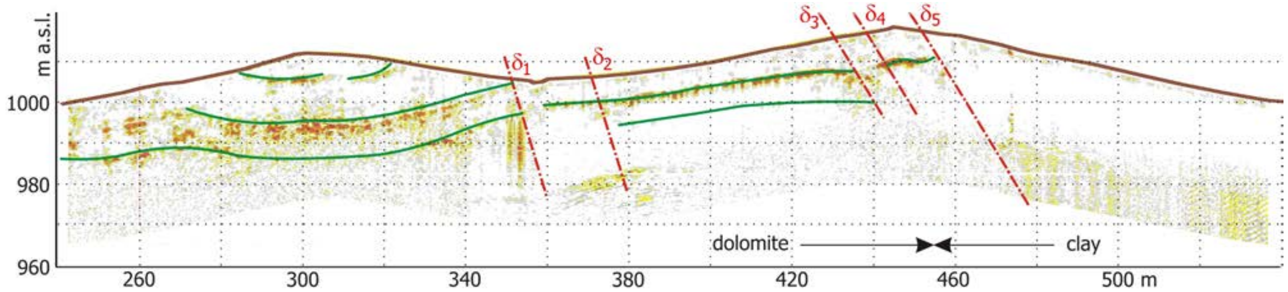


Fig. 6.3.7 Ground-penetrating radar (after Hrubec, 1999)

There is a very low attenuation of the radar signal in the dry carbonate rock and hence reflections can be recorded from depths as much as several tens of metres. In contrast, the radar signal is strongly attenuated in the clay and almost no time signal appears. In the dolomite sequence it is possible to distinguish two distinct boundaries which can be followed along the whole dolomite section. A zone of tectonic disturbance is evident in the interval from 350–375 m on the profile. The right part of the dolomite section is downthrown with respect to that on the left. The displacement may have taken place because of discrete movements on two bounding fractures δ1 and δ2, or as a result of progressive small displacements across the whole width of the zone. It is not possible to decide which of these processes took place solely on the basis of the radar measurements. The radar survey also indicates that the boundary between the dolomite and the clay at the right hand end of the section is probably a fault. Whether there is a single fault δ5, or several smaller faults δ3, δ4 and δ5 is a matter of interpretation. It was not possible to detect any vertical or horizontal boundaries within the clay by using radar.

Another illustration of radar application to detection of a cavity beneath a road is given below. The company Subsurface Evaluations, Inc. in Florida used the SIR 2 apparatus with an antenna system operating on a frequency of 900 MHz to carry out the survey. In Figure 6.3.8, the features of the construction of the road itself are visible, together with a cavity which was formed above a stormwater sewer. On both sides of a cavity, reflections with a paraboloid shape caused by the diffraction of electromagnetic waves are visible. In this figure, the effects of the different velocities of propagation of the electromagnetic signal in the rock environment and in the air-filled environment can be observed. Because the electromagnetic signal is faster in air, the real boundary is "displaced" to shallower depths when the time section is converted to a depth section. If it were possible to introduce a correction for the real velocity of the electromagnetic wave, then the real shape of the cavity at greater depth would correspond with the dashed yellow line shown in the diagram. In fact, the lower boundary of the cavity would lie somewhere between these two extremes, because the fill of cavities is hardly ever homogeneous.

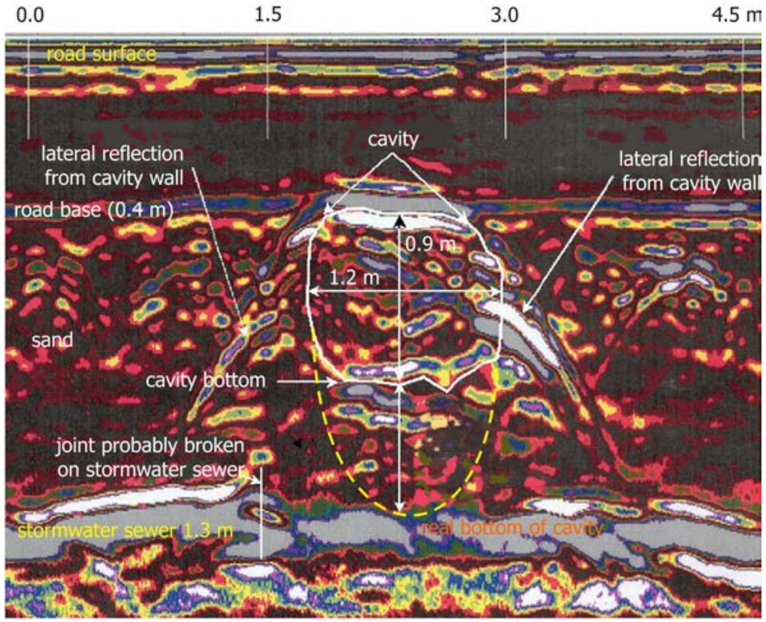


Fig. 6.3.8 Radar detection of a cavity (illustration provided by the company Subsurface Evaluations, Inc.)

It is evident that surveys using radar can only provide partial answers to the problems encountered in the engineering-geological survey of dam sites. Radar is therefore not the ideal tool for investigating the geological structure of an area. This conclusion applies particularly to countries having a humid climate. In arid regions, the situation can be more favourable, though only to a limited degree.

Another use of geoelectrical methods in the survey of dam sites is to study the extent of fracturing in the rock mass, and to determine the prevalent directions of fracturing. For this task, vertical electrical sounding and resistivity profiling, as well as electromagnetic profiling can be used. In all cases, measurements are carried out over strictly limited areas. The use of profiling methods has the disadvantage that the volume measured lies directly below the point at which the measurement is made, but this will not be the same volume all the time. If a description of the rock mass as a whole is desirable, this apparent disadvantage becomes an advantage. When making a survey by VES, the point at which the measurement is made is always the same, but the position of the electrodes supplying the current changes. This means that at every other change in the position of the AB electrodes the volume of the measured rock mass also changes. The interpretation of the measurements can be complicated by a phenomenon called the "paradox of anisotropy". According to this principle, the measurements will show the maximum apparent resistivity following the direction of conductive structures. This results from the fact that the current flowing between electrodes (AB) tends to follow the most conductive zones instead of propagating uniformly in all directions as it would in an isotropic medium.

Both illustrations of the use of this method are taken from the survey of the Slezská Harta site made in the early 1970s. Figure 6.3.9 shows the use of VES, and Figure 6.3.10 the use of SRP. In the VES method, using azimuthal profiles, it is evident that the direction of the most significant fractures in the rock mass changes with increasing depth. Based on geological measurements, S-joints striking 120 ° were prevalent. The frequency of Q-joints was 30 % less. The system of S-joints is clearly indicated by the contour of low apparent resistivity, the axis of which is approximately perpendicular to the axis of the contour of highest resistivity. There is no significant indication of the existence of the system of Q-joints.

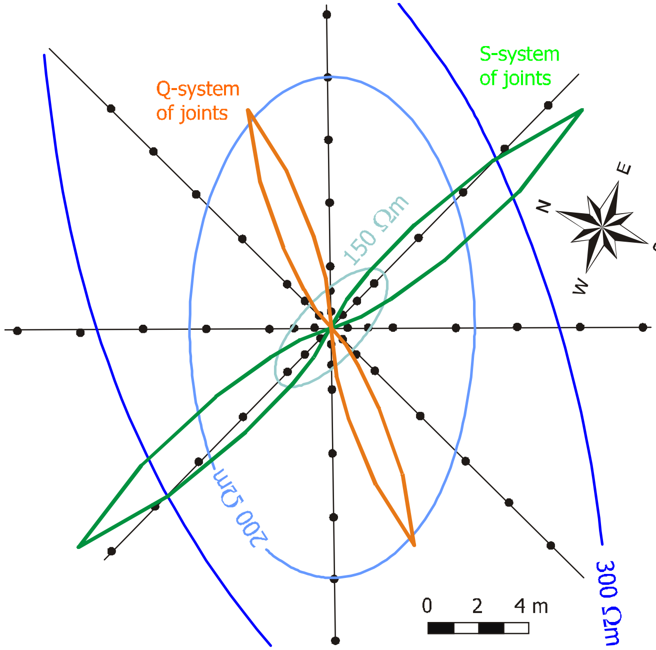


Fig. 6.3.9 Azimuthal VES (after Müller et al., 1968)

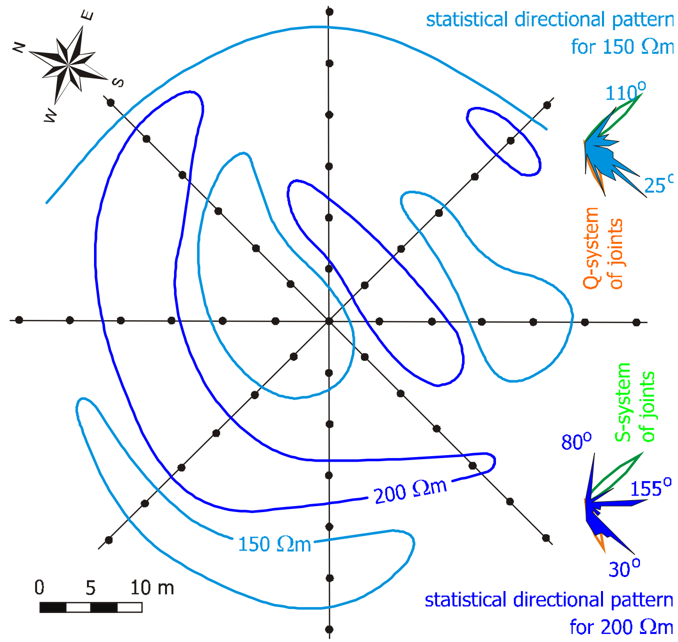


Fig. 6.3.10 Anisotropy based on resistivity profiling (after Müller et al., 1968)

In the case of the results obtained by using SRP, the rose diagrams produced were different for the contours of different values. For the values of low apparent resistivity, distinctly defined directions in the rose diagram show a good correlation with the directions of joints. The directional maximum at 25 ° however does not correlate with any geological structure. Obviously this direction must correspond with the transition between contours indicative of separate directions or to a direction which is partly created by the paradox of anisotropy. These results show that it is more advantageous to base an interpretation on contours corresponding with resistivity minima than on contours corresponding with mean and maximum values. Experience has now shown that when this procedure is used to detect fracture systems within the rock mass, it is more effective to make measurements on a rectangular grid than to use azimuthal profiles. Measurements can be made in two directions and the contours plotted using the geometrical average.

Geoelectrical methods have proved very useful in surveys of dam sites. In engineering-geological surveys, direct current methods are most commonly used, especially vertical electrical sounding and symmetrical resistivity profiling. In the great majority of cases, there is sufficient contrast in the resistivity of different types of rock for lithological boundaries to be clearly defined. Usually, it is not difficult to determine the boundary between rock types by the interpretation of the contours measured by vertical electrical sounding. In nature, there are obviously occasions when the determination of such boundaries is problematic. This arises from the fact that equipment used for geophysical interpretation is not always capable of detecting the smaller variations in the natural rock mass. Modern computer processing has partly eliminated these shortcomings. Further improvements in techniques of interpretation have been introduced through the use of a 2D system that enables entire profiles to be interpreted, rather than individual VES measurements.

Of the resistivity profiling procedures, that most commonly applied to the study of the rock mass in dam design is symmetrical resistivity profiling using the Wenner or Schlumberger configuration of electrodes. Other types of profiling such as combined and dipole profiling are less commonly used. This is due to the fact that forms of profiling based on gradients have more complicated theoretical curves than those obtained by potential profiling. Further advances in resistivity methods have been achieved by the application of multi-electrode systems for measurement. There are undoubted benefits in using these procedures for measurement and processing in cases where there are no dramatic changes in resistivity perpendicular to the section under investigation.

Other geoelectrical methods, often used in geological applications of geophysics, are not so useful in engineering-geological surveys of dam sites. In some cases, special survey procedures can produce very interesting information about the geological structure of an area, but the information may not be essential to answer questions about the suitability of a site for a dam. Of the new methods recently introduced, ground-penetrating radar (GPR) and frequency sounding can be mentioned. It is advantageous to use GPR in situations where it is necessary to decipher the geological structure at shallow depths, or in areas where the moisture content of the rock is very low, especially in the near-surface layers. Definite advantages can be anticipated from the use of frequency sounding. Newly developed methods of measurement and interpretation using frequency sounding have increased the possibility of successfully using this method for the interpretation of deep structures beneath high dams. Geoelectrical methods, however, are less useful than seismic methods for detecting the states of deformation caused by stresses in the bedrock.

6.3.2 Seismic Methods

Seismic methods constitute the second most extensive set of geophysical techniques used in surveys of dam sites. Changes in lithology are not so clearly indicated by variations in the measured values of wave velocities as they are by geoelectrical parameters, but provided that a dam site is not located on rock units with very different properties, the magnitudes of the velocities of both longitudinal and transverse waves are mainly a function of the physical-mechanical state of that rock mass. It will always be necessary however to take the lithology of the medium into account when interpreting seismic measurements.

When geophysical methods were being introduced for engineering-geological purposes during the 1960s and 1970s, the equipment was that used by teams dealing largely with mineral exploration. The application of individual geophysical methods to engineering-geological problems depended on the expertise of the teams and the availability of appropriate instruments. The use of geophysical methods was also governed by the cost of carrying out a survey. In the former Czechoslovakia, geoelectrical methods were particularly favoured; in Poland, gravimetry was widely used.

In seismic surveys of dam sites, longitudinal waves are used almost exclusively. Transverse waves are applied sporadically, only when it is particularly important to know the mechanical properties of rocks, but never for the survey of a geological structure. Surface waves are used even less. Of the seismic methods, shallow seismic refraction is by far the most widely used. Seismic reflection is used much less because it does not yield satisfactory results at the depths of interest. The use of seismic methods in 'borehole – surface' combinations has increased in recent years. The use of such configurations for seismic measurements has been enabled by advances in tomographic processing of results, notably taking into account the curved paths of propagation of a seismic signal.

Classical methods for processing shallow seismic refraction data proceed from the theoretical assumption that the value of velocity is constant below the refraction horizon, and that the boundary is plane and only moderately inclined. These prerequisites, however, are not usually fulfilled. The superficial layer, formed as a rule by silty clay, debris or intensely disturbed material, passes into weathered bedrock. The degree of weathering decreases as depth increases and intact rocks are found only at greater depths. The vertical transition in velocity between individual types of rock medium is usually smooth. When processing the data obtained by shallow seismic refraction in such areas, it is usually difficult to determine the refraction points of travel-time curves. The inaccuracy in this determination results in the inaccuracies in the determination of both the depths and velocities of individual layers.

The use of computer applications in geophysics has made possible new methods of data processing based on theoretical techniques. Improvements in the algorithms used for computer processing have enabled much better interpretation of results. Earlier methods of interpretation of shallow seismic refraction data, i.e. the method of critical distances and the Hagedoorn method, proceeded from simple theoretical assumptions. The method of critical distances (otherwise called the method of the refraction point) even assumes the constancy of velocity of a refracted wave along the refraction horizon. Another prerequisite is the planar configuration of the refraction boundary. This method enables information about the depth of the refraction horizon and the corresponding velocities to be obtained. The method is capable of providing information about several refraction horizons beneath one another. The determined refraction velocity is the average of the complete section of a specific travel-time curve. An advantage of these methods is their simplicity, and hence the speed with which it is possible to obtain information about the studied rock medium.

The method t0 (otherwise called the plus-minus method or the Hagedoorn method) allows for certain smooth changes in velocity along the refraction horizon. Small changes in the relief of the refraction boundary are also possible. The radius of the curvature of bumps (irregularities) on the boundary should be much greater than the amplitude of the irregularity. This method thus gives information about the depths of a boundary below each sensor used in the field survey. It holds true that velocities are mostly determined from certain sections of travel-time curves or by the moving average. The dimensions of such sections are governed by real velocities in the bedrock. Certain inaccuracies do originate when applying this method to cases where marked changes in the shape of the refraction boundary occur.

Both the methods described assume constancy of velocity in the vertical direction below the refraction horizon, i.e. they do not allow the possibility of penetration by a seismic wave. New interpretation techniques have helped to eliminate these disadvantages. The current trend follows two approaches. One approach is to eliminate errors caused by irregularities on the refraction boundary and the other approach is to reduce errors caused by the penetration of a seismic wave.

The first solution is to use the Palmer method, in which a medium is considered to be composed of two layers. It is possible to eliminate the effect of large irregularities on the refraction boundaries by using a suitable procedure and by choosing the velocities in the cover. This method is suitable for circumstances where the overburden is thick, in the order of 20 to 30 metres. The effect of deviations that this method is designed to eliminate is also dependent on the ratio of the velocities of the direct and refracted waves. If this ratio rises, then the effect of irregularities in the refraction horizon is reduced.

The second approach depends on the elimination of errors produced by the penetration of the seismic wave through the bedrock. This method of processing takes into account the increase of velocity below the main refraction horizon. This is chiefly determined by the geological structure, the knowledge of which is important for solving engineering-geological problems. Results obtained using this method can be used to determine the thickness and velocity of the first layer and velocity changes in the bedrock. Velocity changes below the refraction horizon can show different patterns and these can be described in mathematical terms as different velocity laws. In the literature, there are references to linear, exponential and parabolic velocity laws. In the opinion of the authors, the above-mentioned laws have a specific disadvantage, namely that velocities increase steadily with increasing depth. However, up to the present time, only the linear law of increase in velocity has been used for practical purposes. It has the advantage that it is relatively simple theoretically and can be transformed easily for computer use.

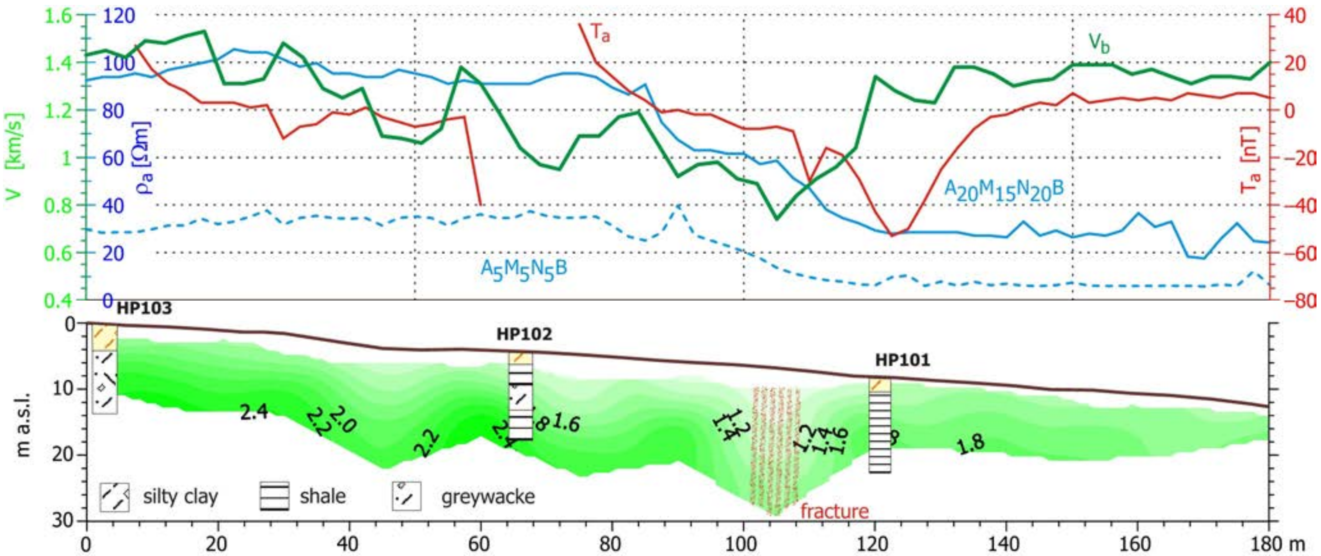


Fig. 6.3.11 Fracturing of the rock mass indicated in the field of velocities measured by SSR

An illustration of the use of shallow seismic refraction in distinguishing between disturbed and undisturbed rocks has already been presented in the preceding chapters (Fig. 2.3.25). In the case of the Genal site, it was used to detect fracturing of the rock mass caused by a slope failure. Figure 6.3.11 is an example depicting a fracture zone cutting a rock complex at the Fulnek site. The area of the site is underlain by a sequence of Culm sediments consisting of clayey shale and greywacke of the Hradec-Kyjovice Formation. The greywacke is fine-grained and thick-bedded, whereas the shale is finely laminated and shows graded bedding. The thickness of the shale beds reaches up to several metres. In the area surveyed, the beds of the Culm formation form an anticline with an axis in the NE–SW direction. Open tectonic fractures are present in the greywacke at the site.

In the field of velocities, the main feature is the decrease that occurs in the interval between stations at 100 to 112 metres. At a depth of about 10 metres, the velocity decreases from 2.4 km/s in the intact mass to 1.1 km/s in the fracture zone, which is a velocity characteristic of debris rather than intact rock. Neither in the curves obtained by resistivity profiling nor in the results of vertical electrical sounding, are there any indications of the fracturing of the rock mass. Based on the results of the seismic survey, there is evidently a change in the lithological composition of the surveyed area corresponding to the measured changes in resistivity. According to the results of drilling, shale occurs at greater depths, while greywacke is present at the top of the section. This is not typical of the Culm sedimentary sequence in northern Moravia. By comparing the results of all the geophysical measurements, the conclusion was reached that the weakened zone in the rock mass was due to the effects of brittle fracturing. This conclusion is supported by the observation of open fractures in a nearby stone quarry. Because the fracture zone shows no decrease in resistivity, it is concluded that the fractures have not been filled by silty clay or clay and that the groundwater has a relatively high resistivity, indicating that it is not significantly mineralized.

In a number of cases, seismic methods have revealed some unusual geological phenomena. At the Žamberk site in the Bohemian Cretaceous, a survey along a short profile was made. An interesting feature is the distinct velocity maximum shown on the right side of this profile (Fig. 6.3.12). This maximum is caused by the intense silicification of the argillaceous sediments in the Cretaceous sequence. The existence of this strong silicification was also confirmed by drilling. It can be proved by detailed analysis of velocity contours that this zone is located close to a fracture zone at shallow depth. The question remains as to whether the silicification is related to the fracture zone itself or not. Another possible scenario is that the original fault was healed by quartz and that, during reactivation of movement on the fault, the rock mass around the healed zone was fractured. This question could only be answered by making additional measurements and direct geological observations. In this case, it was necessary to decide whether the answer to this question was essential for the successful outcome of the engineering project. Weathering of the rocks along this profile reaches a depth of about six metres.

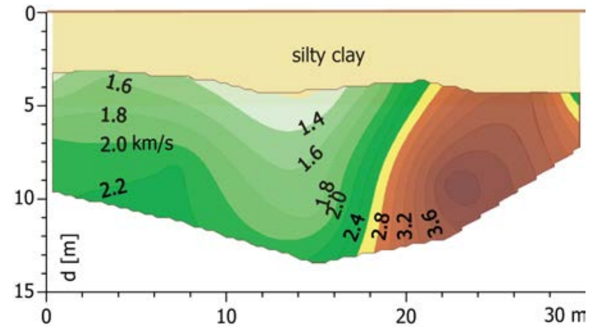


Fig. 6.3.12 Silicification of claystone at Žamberk

Figure 6.3.13 shows an example of measurements made for the purpose of determining the velocity gradient of the rock medium near the ground surface. Field measurement was carried out by detonating two "shots" outside the array of geophones. Velocities below the refraction horizon are displayed because the aim of the measurement was to provide a picture of velocity changes within the pre-Quaternary bedrock. In the section, the pattern of velocities of longitudinal waves is shown and the interpretation of fracturing in the rock mass is based on this. Curves of the following form were interpolated using the individual values of seismic velocity:

$$V_b = a_1 + a_2h + a_3h^2$$

The curves show that velocities rise in the block of intact rocks (at station 30) and in the block of fractured rocks (at station 75). It is clear from this that the use of the linear velocity law has not produced ideal results. This illustration, as well as the results of other surveys, suggests that it would be beneficial to renew theoretical work to develop procedures based on non-linear growth of velocities. Only after the derivation of suitable models will it become possible to proceed to the stage at which the seismic data can be processed by computer so that these problems can be solved routinely in practical surveys. The introduction of multi-channel devices for field measurements will provide the large amounts of data required to solve geological problems using these new procedures.

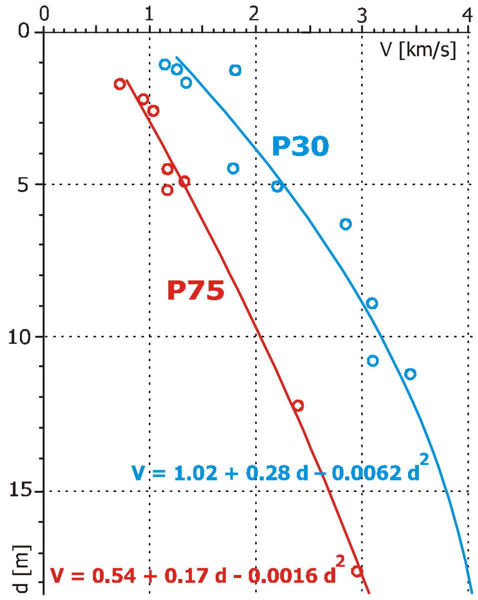


Fig. 6.3.13 Change of velocity with depth

The advances in instrumentation and computer technology are also reflected in the development of methods for processing radiography. One of the new trends in processing geophysical measurements is the use of tomography. In theoretical terms, there are two types of tomography used for processing data obtained by seismic radiography: tomography along straight paths and tomography along curved paths. Tomographic processing of radiographic data is used for both seismic and electromagnetic radiography. These types of geophysical measurements, however, also require specific tomographic procedures. Electromagnetic radiography can be processed using the assumption that the paths of propagation of signals between transmitter and receiver are straight. Such processing is relatively easy as regards mathematics and software. Seismic radiography requires a more complicated mathematical solution. A seismic wave propagates along the shortest-time path, i.e. not along a geometrically straight line. Tomographic processing of seismic radiography, however, gives qualitatively better results than classical processing in the form of wave diagrams.

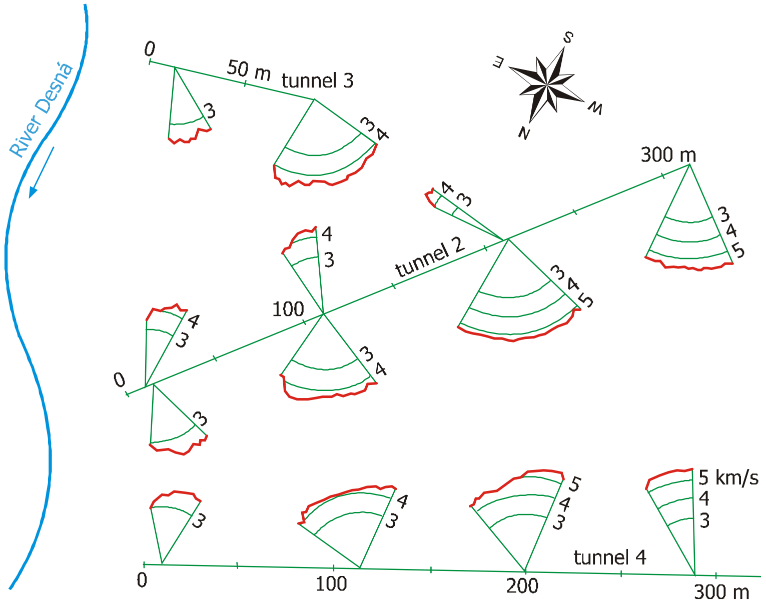


Fig. 6.3.14 Seismic radiography - fan graphs

The survey for the Dlouhé Stráně pumped storage hydroelectric plant (PSHEP) provides an illustration of the use of tomographic processing of radiography between tunnels. The original seismic radiography was carried out in 1973. The measurements were processed then using the classical method of fan velocities (Fig. 6.3.14). To illustrate the differences in processing by classical and new methods, the results produced by seismic tomography are presented in Figure 6.3.15. Seismic radiography was carried out between three tunnels. The spacing chosen for the geophones was ten metres and the spacing between blasts was about a hundred metres. The classical processing by means of fan graphs provided useful data on the state of the rock mass at the time of measurement. These results provided a basic picture of the changes in velocities in the rock mass. It was established that velocities deeper inside the mass are higher, which suggested that it would be possible to build a cavern for an underground powerhouse in that area.

Figure 6.3.15 shows that the tomographic processing of the results of seismic radiography yields a qualitatively better result. At first sight, it is evident that the geotechnical quality of rocks inside the rock mass is markedly higher. In addition to this basic observation, the velocities plotted tomographically provide other useful information. The most significant is the evidence that the rock mass can be divided into two megablocks. The first is an extensive zone around the Desná Fault, which covers the eastern part of the ground. The second lies on the opposite side of the surveyed ground and is a megablock of relatively unfractured rock. In addition to this basic division into two units, it is also possible to identify other zones of low velocity which correspond to zones of strong and continuous fracturing of the rock mass. The ability to detect zones of continuous fracturing was essential for the identification of individual fracture zones. It is only with difficulty that individual

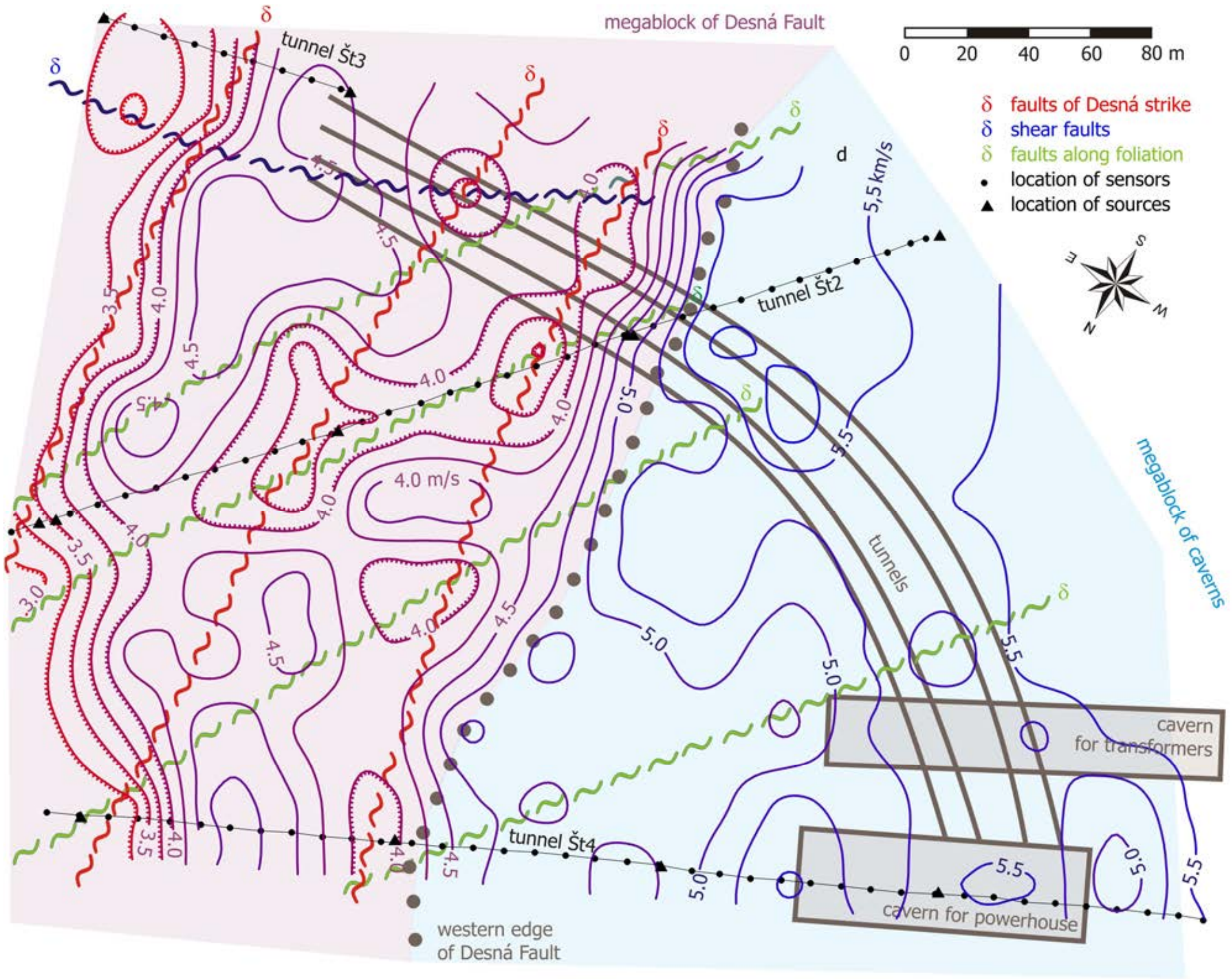


Fig. 6.3.15 Tomographic processing of seismic radiography, Dlouhé Stráně PSHEP

fracture zones could be correlated using the geological description of underground sections. Based on the tomographic processing of the results of seismic radiography, three principal directions of tectonic fracturing were identified: one corresponds to the Desná Fault, a second to fractures parallel to the strike of bedding in the rock mass, and a third to a set of shear fractures. The latter are abundant within the Desná Fault Zone and account for the pervasive fracturing of the rocks that was caused by local shear movements.

Another example is the survey carried out at the Žilina dam site between boreholes JŽ401 to JŽ403. The section shows that the rock mass can be divided into three sub-horizontal blocks (Fig. 6.3.16). They are marked in the figure as blocks A, B and C. Block A corresponds to the Quaternary cover of the surveyed area. The lower contact is marked by a sharp growth of velocities. This gradient is not so marked in the places where the rocks in block B are fractured. The boundary between the Quaternary sediments and the Palaeogene bedrock is marked by a velocity of approximately 1.0 km/s.

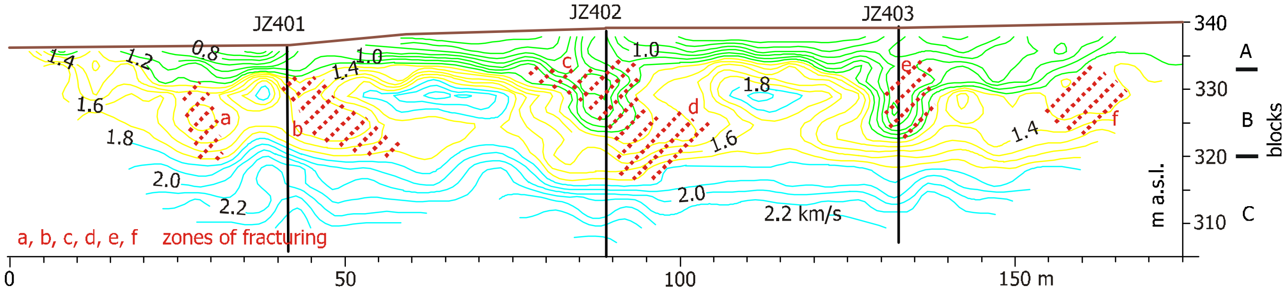


Fig. 6.3.16 Seismic tomography between boreholes at Žilina

Block B is the block with the greatest contrast in behaviour of the rock mass. The values of velocities change from 1.0 to 2.0 km/s. Local maxima and minima of velocities occur frequently. This block shows properties matching Palaeogene sediments that have been intensely fractured, and in parts of which the original stress has been relieved whereas in other parts stress remains concentrated. The base of this block can be defined at a level of about 320 metres above sea level. The contours show that in certain places fracturing extends another five metres deeper. The intense fracturing of rocks detected in this section is due to the fact that the profile lies perpendicular to the tectonic boundary of a belt of klippen and the fracturing is caused by the tectonic stresses focused along this boundary.

In the surveyed section, it is possible to identify four sub-vertical zones where weakening of the rock mass has taken place. Evidence of fracturing in borehole JŽ-403 was also demonstrated by the decrease in recovery of drill core at a depth of 12.5 metres. Another explanation for the weak zones b, c, e, and f is that strata with weaker mechanical properties occur within the Palaeogene sequence. Beds b and f could then be one layer and beds c and e a second layer. In nearly all the weakened zones, there are places where velocities increase distinctly, forming closed maxima. In some places, the increases are so large that they match the velocities of the rocks up to 15 metres deeper.

Block C corresponds to the undisturbed Palaeogene deposits. It is notable that only fracture d shows evidence of continuing to greater depths. Evidently, the layer of Palaeogene sediments is subject to strong horizontal stress near the main fault in this valley and thus most of the fractures are tightly compressed.

Seismic tomography enables a range of different observations to be made on a geological section which could not be obtained otherwise. Measurements can be carried out both in borehole – borehole mode and borehole – surface mode, but modern procedures enable surveys to be carried out using only surface measurements as well. Modern developments in the techniques of measurement and processing of information obtained by seismic tomography have enabled surveys to be made of man-made structures such as dam walls and subsoils as well.

The final illustration of the use of seismic tomography is not a survey of rocks in a geological section, but a survey of the body of the Pontevedra dam. The body of the dam was lined with variably weathered granite blocks and the dam itself is founded on underlying granite. A tomographic section was made in a vertical plane perpendicular to the long axis of the dam. The positions of the sources of seismic waves and the seismic sensors used for this tomographic radiography are shown in Figure 6.3.17. Based on the contours of seismic velocities the dam body and subsoil can be divided into four domains. The first is a zone of low velocities just below the dam crest (especially below sources I and II). This is evidence of discontinuities in the dam body below the screen. In addition to this decrease of velocity in the screen, the dam is affected by the stress within the triangular block of the superstructure on the downstream side of the dam. Tensile stresses, to which this part of the dam is exposed, cause a further decrease of velocities. Therefore, the zones of lower velocity appear in cells below source III and below sources I + II. The horizontal gradient of velocities between sources III and II indicates that the highest tensile stresses are located in this part of the dam wall. This is obviously the reason why longitudinal cracks formed in the dam wall during reconstruction.

In another section of the dam at a level corresponding to sources V to VIII, the velocities of longitudinal waves vary from 2.0 to 2.6 km/s. These velocities are in the normal range of velocities of longitudinal waves that would be expected in the dam. The third significant feature in the pattern of contours is a zone of higher velocities in a horizontal strip at the level of sources IX and X. This can be identified as a zone in which stress is concentrated. It forms an arch above a zone of distinctly lower velocities in the lower third of the dam. Most important is the zone of low velocities between sources VIII and IX. The decrease of velocities to below 2.0 km/s suggests that tensile or shear stresses are present within the body of the dam. The effects of this stress were possible to observe when a visit to the reconstructed dam was made by the Geotest team in the summer 1994. More than a year after the reconstruction of the dam body, seepage through the dam at this level was observed. This is shown in the photograph in Figure 6.3.17.

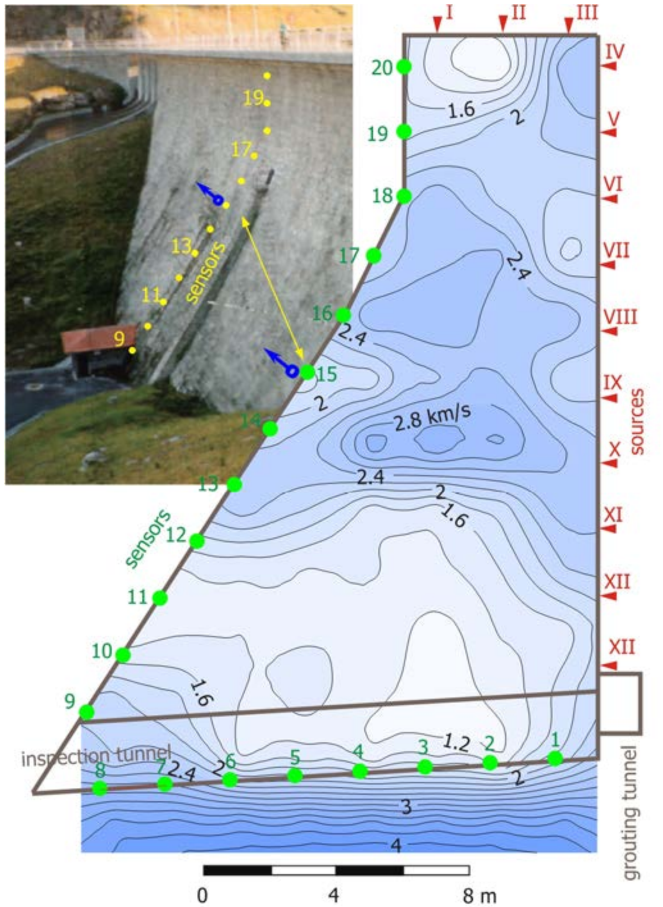


Fig. 6.3.17 Tomographic check of the dam body at Pontevedra (a photo by P. Bláha - 1994)

Next, there is a wide zone of low velocities extending downwards in the profile below source XI to the inspection tunnel. Here, contrary to expectation, the velocities of longitudinal waves are low. There is a visible difference in the quality of the masonry between the lower and upper part of the dam on the downstream side. This also explains the decrease in velocities in the lower part of the dam. It is reasonable to assume that the actual fill of the dam body, as well as the outer lining, would be of different quality. Seepage through the dam and washout of the binding material must have been more prevalent in the lower part of the dam than in the upper part. The zone of high velocities in the dam subsoil is the last notable feature of the tomographic profile. At first sight it appears that the granite massif has better mechanical properties than the dam body, however the absolute values of velocities (3.6–3.7 km/s) show that the granite outcrop is also weathered and that the rocks have rather poor geotechnical properties.

Certain special problems in civil engineering, geotechnics and engineering-geological surveys for dams can be solved successfully by means of ultrasonic measurements. This method is particularly applicable to surveys of small volumes of geological or man-made material. This is because of the rapid attenuation of high-frequency elastic waves in natural geological media and man-made materials. Ultrasonic measurements can be used effectively to determine the mechanical parameters of a studied medium for applied purposes. Tests can be carried out on various samples of construction materials and drill cores and, in particular, ultrasonic radiography of small volumes of natural geological material can also be carried out.

There is a wide variety of apparatus that can be used only for making ultrasonic investigations. The use of watertight heads and heads with a conical shape permits flexibility in field and laboratory applications but the basic problem of how to obtain a complete understanding of the geological medium still remains. Classical heads can be used only for studying samples prepared by cutting two parallel plane surfaces. Conical heads can be used for measuring bulk rock samples, which means that it is possible to measure almost all types of cores and, in addition, to measure the anisotropy of samples in different directions. Using these heads it is also possible to carry out radiography of larger bodies. If the bodies consist of rock, it is possible to carry out radiographic measurements of sections up to decimetre dimensions. In concrete bodies, the penetration of radiographic measurements may be a few metres. Watertight heads enable measurements of the material below the water surface. It is possible to substitute conventional sonic logging by discrete measurements made using the “point sonic logging” technique.

The first illustration is the ultrasonic measurement of drill cores from borehole J1 on an old dam body at Jevišovice (Fig. 6.3.18). The first twenty metres of the hole was drilled through the masonry of the old dam, and the rest was drilled through orthogneiss in the dam subsoil. There is an obvious difference in the character of both the media in the graphs of velocities measured along the borehole axis (red curve) and perpendicular to the borehole axis (black curve). The velocities in the dam fluctuate significantly and are quite different in each of the measured directions. In the orthogneiss massif, after an intermediate layer at depths from 20 to 23 metres, there is a more or less continuous increase of velocity with depth. The velocities in both the measured directions are similar and the pattern of increase is more or less the same. The velocities of longitudinal waves change from 4.0 km/s (24 m) to 4.5 km/s (33 m). Whereas the anisotropy of velocities in the vertical and horizontal directions in the orthogneiss is rather small, the anisotropy of the material of the dam body is distinct. The fluctuation of the absolute values of measured velocities in the dam wall is also greater. This shows that the material used for the masonry of the dam was heterogeneous. It is likely that these variations in the mechanical properties of the material were not so evident at the time when the dam was constructed, but that since the rock was incorporated in the dam body, it has been exposed to the effect of processes which have caused significant changes in its mechanical properties.

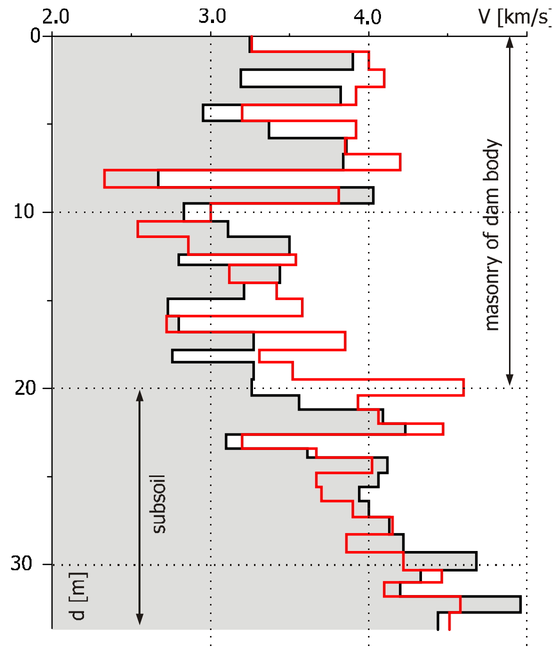


Fig. 6.3.18 Ultrasonic measurement on drill core

Measurements of drill cores record the state of the rock at the time of measurement. Change in the moisture content of drill cores and the relief of stress in drill cores after they have been recovered from a borehole lead to decreases in the measured velocities of longitudinal waves. It should be noted, however, that the comparison of velocities measured in a relieved sample of drill core and in rocks *in situ* in a borehole under natural conditions of stress and moisture content in their original stratigraphic and structural context can provide very valuable information about the state of the rock mass.

An illustration of the results of point sonic logging is shown in Figure 6.3.19. These measurements were made as part of a survey for the Budišov dam project. The measured values show how complicated the pattern of velocities along the borehole axis is. The relatively high velocities just below the surface are probably caused by cement filling in the cracks when preparations to mount the conductor casing were made. These higher velocities are followed by a decrease down to values just above 2.0 km/s. This pattern is repeated several times and indicates that the geological structure is complicated by tectonic fracturing of the rocks. Narrow intervals at depths of 7–15 metres over which velocities could be measured show that the frequency of fracturing is greater at the top of the Culm sequence than at greater depths. The measurements show that much more detailed relationships can be resolved when measuring small volumes of a surveyed area. If a lower frequency is used for making seismic measurements, small-scale local inhomogeneities cannot be detected through changes of velocity. The measured curves are, therefore, smoother and can create a misleading impression if the observer is not experienced in this type of work.

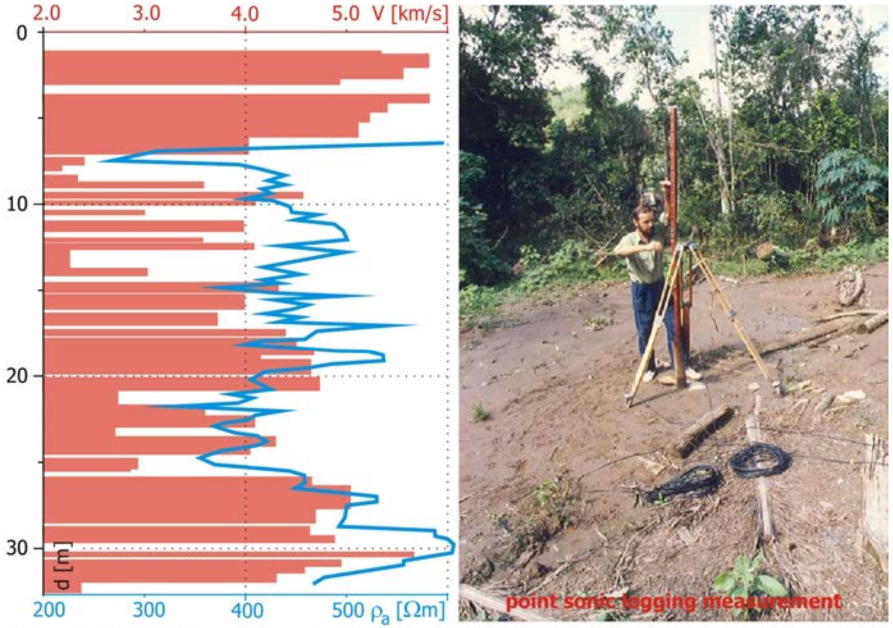


Fig. 6.3.19 Ultrasonic log of a boreholes (a photo by M. Vlastník - 1988)

Ultrasonic measurements can be successfully applied for checking the integrity of concrete structures forming part of a dam wall or ancillary facilities. In concrete bodies, it is possible to obtain a useful signal from distances even greater than five metres. The illustration in Figure 6.3.20 shows the results of radiography of concrete buttresses about 1.5 metres in diameter by using conical ultrasonic heads. Measurements were always made in 17 directions, that is, 11 ° arcs of the buttresses structure were measured successively. The curves of velocities obtained in three buttresses show that the concrete in the buttresses is isotropic. The velocities measured in individual arcs do not differ by more than 0.7 %. No substantial differences in the quality of the concrete used to construct the individual buttresses were detected; the maximum difference in the velocities measured in the separate buttresses was 0.24 km/s.

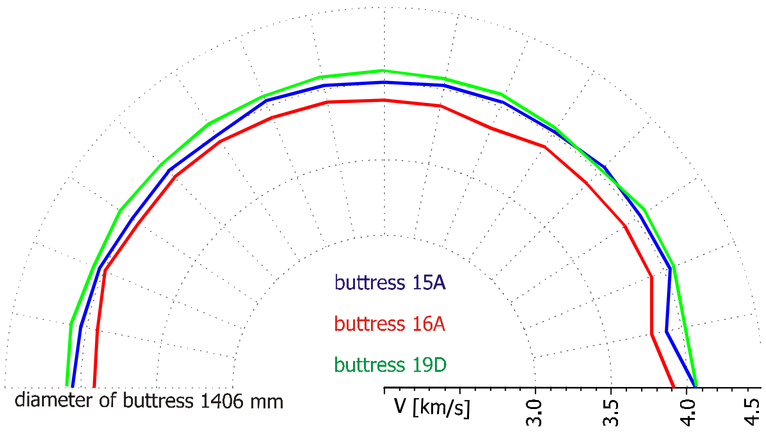


Fig. 6.3.20 Ultrasonic check of concrete buttresses

Another example of the use of conical heads for making ultrasonic measurements is a shear test carried out on a sample cube for the purpose of rock mechanics in the Viola 2 tunnel in Hrhov. The cube, with edges 70 cm long, was radiographed in both horizontal directions at ten-centimetre intervals so that measurements were made on a square grid.

Figure 6.3.21 shows the velocity contours from one direction. It is apparent that the material is relatively homogeneous with velocities ranging from 3.65 to 3.95 km/s. A definite zone of weakness can be seen in the centre of the test cube. The velocity values are lower than would be predicted for the massive limestone in which the Viola 2 tunnel was driven. Before the measurements were made, the anticipated velocities were expected to be 0.5 to 1.0 km/s greater. The reduction of velocities may have been caused by relief of the stresses on the test cube, and/or by its partial desiccation.

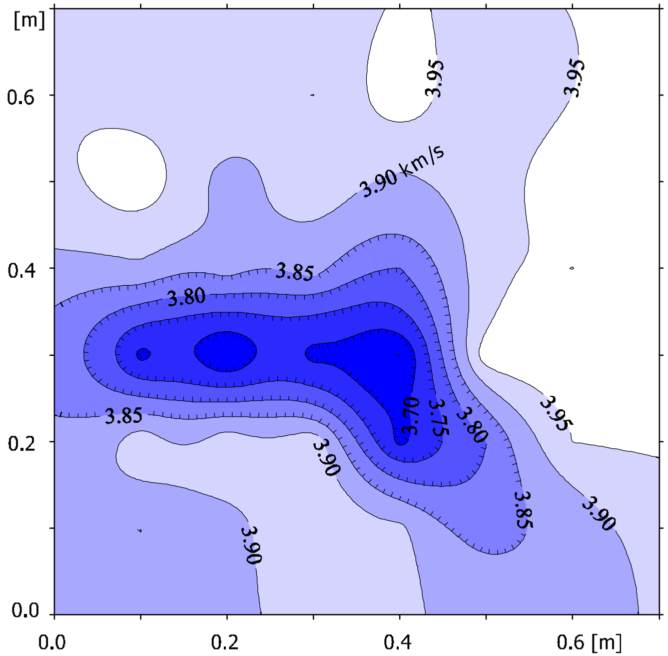


Fig. 6.3.21 Ultrasonic measurements made on a test block of limestone

Seismic geophysical methods were once the second most important group of techniques used for the survey of dam sites, though today they are perhaps the most important. Their applications can be divided into two broad groups.

The first application is for the identification of discrete boundaries in the physical properties of the geological formations and structures underlying a dam site. The method most commonly used is shallow refraction sounding. In interpreting seismic measurements, it is necessary to pay great attention to the statistical processing of the measured velocities. Statistical evaluation provides a basis for the further understanding of processes which have taken place in the rock mass. Recently, surveys by shallow seismic reflection are being increasingly used. Experience shows that the best results are obtained by the use of transverse waves. They can be used to identify sub-horizontal and inclined geological boundaries. Another notable possibility is the use of seismic reflection to define boundaries between different media that show a characteristic inversion of velocity.

The second application of seismic methods is for identifying the state of deformational stress in a dam site and its vicinity, including the determination of the physical and mechanical properties of prepared rock samples as well as those of rocks *in situ*.

6.3.3 Logging Methods

Measurements made by logging in boreholes are used less frequently than surface geophysics in the survey of dam sites, but they provide valuable information that can be applied successfully in guiding the engineering and construction of a dam. Therefore, the same attention is given to logging as has been given to geoelectrical and seismic methods. Logging provides information about the physical properties of the rock mass under investigation. Therefore, the values of physical properties measured by logging provide a necessary basis for geotechnical calculations and can subsequently be applied for the interpretation of the results of surface geophysical surveys.

The correct and thorough interpretation of logged data can provide information about:

- The geological profile penetrated by the borehole;
- The physical and mechanical properties of rocks;
- The permeability and porosity of rocks;
- The elevation of the groundwater table and capillary fringe;
- The character of fracturing in the rocks around a borehole;
- Zones with high stress concentrations; and
- The state of the borehole on completion (cementation around the collar, casing, etc.).

The value of the information that can be obtained by logging is not always fully appreciated by engineering geologists and geophysicists. The interpretation of logged measurements must start with a knowledge of the geological formations and tectonic processes that govern the environment at the site in which the rocks are found. By using the results of logging it is, however, possible to deduce facts about the geological composition and structure of a dam site which were not known before the survey was carried out.

The first illustration of the use of logged measurements in the survey of a dam site is that carried out at Lubina during the late 1970s (Fig. 6.3.22). Sonic logging was not included in the set of methods used because at that time a probe suitable for narrow boreholes was not available in former Czechoslovakia.

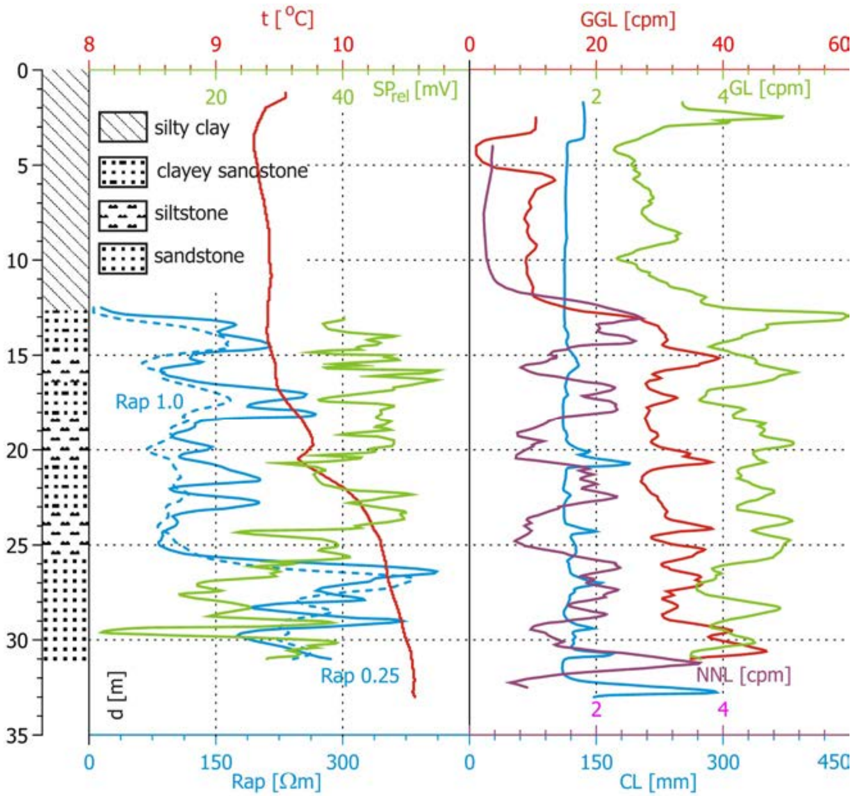


Fig. 6.3.22 Geophysical logs of borehole J51 at Lubina

A surprising feature revealed by the logged measurements is the great thickness of the Quaternary cover produced by fracturing and deep weathering of the underlying sedimentary complex. At greater depths, there is an alternation of siltstones and clayey sandstone beds, and at the bottom of the borehole, a layer of thick greywacke was intersected. From the pattern of the CL and GGL curves, it can be concluded that the beds of siltstones were somewhat weakened, but a distinct belt of fractured rock was not intersected anywhere in this borehole.

One of the latest possibilities of obtaining information about discontinueties along the axis of a borehole is the use of an acoustic televiewer (acoustic borehole-wall imaging - ABI). This type of instruments enables us to obtain an idea of the bedding of the environment, foliation and the occurrence of fractures, and also to determine their strike and dip. The apparatus for this measurement contains an acoustic transmitter that irradiates the borehole wall through a rotating reflector and simultaneously records the reflected signal. A drill probe records the amount of reflections obtained from the borehole wall during the rotation of the reflector (about 100 – 300 points per revolution). During a slow movement of the probe, the reflection from the borehole wall is recorded every several millimetres. This fact, together with perfect orientation of the probe, enables us to obtain a precise idea about the diameter of a borehole and about the reflectance of every reflex face. During measurement, the borehole diameter and the intensity of the reflected signal are recorded. Both parameters are ultimately plotted as two developed planes (Fig. 6.3.23). By in-house processing, it is possible to further obtain an "outer" view of the borehole together with the depiction of the power of the signal, i.e. with a certain characteristic of the quality of the rock environment on the borehole wall. In addition, it is possible to construct a perpendicular section through the borehole at any point in the borehole, showing its exact shape. Another possibility of processing is to determine the parameters of planar features of particular discontinuities along the borehole axis.

Figure 6.3.23 shows an example of such measurements in a depth interval of 31 to 41 metres in borehole PZV-1 at the Pozďátky site. The left column of the figure gives a developed (unfolded) record of the borehole

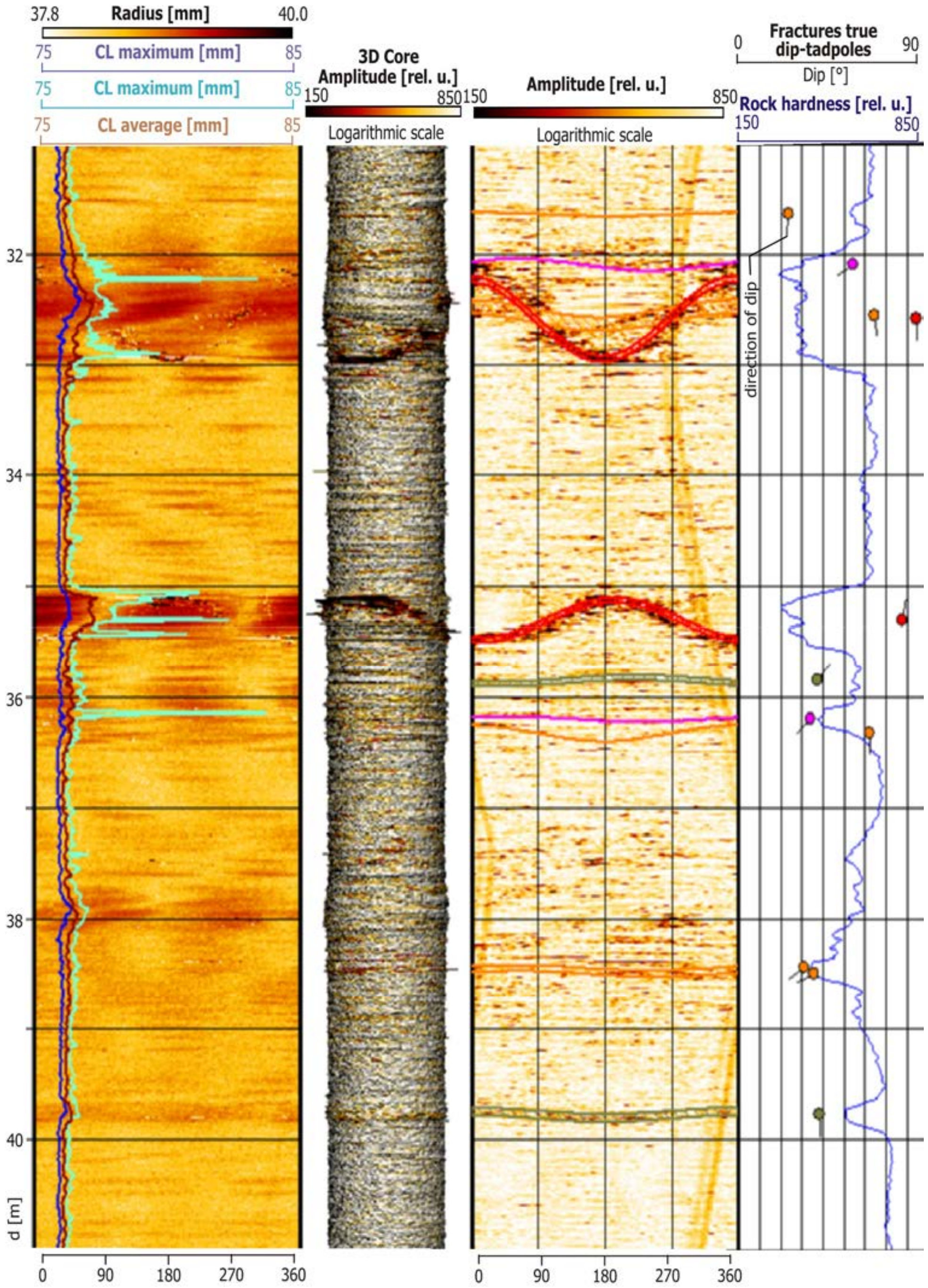


Fig 6.3.23 Acoustic borehole-wall imaging (after Pitrák, 2011)

diameter, including the curves of the minimum, maximum and mean diameters of the borehole. The second column represents the outer view of the borehole, depicting the strength of the reflected acoustic signal from every measured point. In a computer, it is possible to rotate the image of the borehole and watch its shape from any direction. The assigned colour indicates mechanical properties of rock. The darker the colour is, the more fractured the rock is. A better description of fracturing together with a depiction of planes of particular discontinuities is given in the third column, in which every plane of discontinuity is shown as a sine curve. The steeper the discontinuity is, the larger amplitude of the sine curve is. The last column depicts a curve that characterizes strength parameters of rock. The scale of the curve is in relative units and the scale itself is logarithmic. In addition to this curve, the last column also gives data about the dip and downdip direction of individual planar surfaces. Every colour-coded discontinuity in the third column is assigned a colour point, the position of which gives the dip angle. The short line segment at the point depicts the downdip direction, with the north being up and the south down.

Another possibility of obtaining a perfect view of drilling results and the rock mass is the use of a new generation of televiewers (the so-called optical borehole-wall imaging, OBI). These no longer provide an image of a borehole in the axial view or a section in the radial view, but an image of a borehole as a developed surface of a cylindrical area (similar to the ABI method). In contrast to the ABI system, televiewer systems often use lenses with a very short focal length (fish eye). The television image is then computer-processed up to the resulting form of a developed cylindrical area. The actual resolution of such devices is usually one millimetre. A combination of a TV recording and an inclinometric measurement enables data on the spatial orientation of planar features of the geological structure to be obtained. In addition, the TV image clearly shows defects in a borehole (borehole breakout) and a fall-out of fragments, if any, from the borehole wall. The ability of these instruments to describe the borehole wall starts with cutting diameters of wireline drilling and ends with diameters of roughly 250 mm. A great difference between the ABI and OBI systems is in the requirements for the state of the borehole. Whereas the system of acoustic televiewers better works in boreholes filled with water or another liquid, the system of television documentation is more successful in dry boreholes or boreholes filled with clear water.

An example of the results obtained using measurement by the OBI system of the company Terratec is given in Figure 6.3.24. In the upper part of the figure, there are examples of developed TV recordings of different lithological types from borehole sections about 100 cm long. The columns most left show intense fracturing of limestone, including signs of karstification. The second column shows open horizontal joints and a closed oblique joint (roughly in the centre of the figure). The third example of limestone is from a relatively undisturbed environment with intercalations of exotic material. In granites it is possible to see an open subhorizontal fracture (at the top) and two steep joints in the lower part (a characteristic sine shape). The upper steep joint is crossed by another subhorizontal fracture. On the sandstone of atypical colour, there are visible individual bedding planes and a fracture roughly of the same dip as the bedding planes. The flysch sediments show various sizes of grains that form the sediment, as well as their different lithological origins.

The lowest part of Figure 6.3.24 shows one of possible statistical evaluations of the recording. The downdip directions are projected in the lower hemisphere. The statistical evaluation can be processed for the whole borehole or only for its individual intervals. In addition to this approach, it is also possible to choose similar processing as in ABI.

A more complicated case is that of logging measurements made in boreholes drilled using wire-line techniques, when measurements using nuclear logging methods must be made inside drill rods for safety reasons. In borehole MEL-3 (Fig. 6.3.25) in the granite massif at Melechov, distinct fractures were verified by GGL and NNL methods by making measurement in drill rods. After these rods were pulled, the borehole was impassable at greater depths for other probes. A significant fracture at 124 metres was also confirmed by subsequent hydrogeological measurements, when it was proved that the only water flowing into the borehole came from fractured beds in an inaccessible part of the borehole. The yield was relatively high, amounting to 0.2 l/s, and was accompanied by a 2.2 metre drawdown of the water table.

It is necessary to point out certain differences between the GGL and NNL curves. When comparing a core of about 5 cm in diameter and the rock volume which is measured by an NNL probe, it is necessary to take into account that data obtained by logging are generated from a volume about 20–30 times larger on average. The GGL probe is maintained in close contact with the inner wall of the rod and measures in a narrow sector to a distance of roughly 20 cm because the gamma source is collimated and shielded to minimize the adverse effect of the borehole diameter. Due to the collimation of measurements, the asymmetry caused by cavities is also reflected in the GGL curve and this can be significant, especially in granite. This also explains the

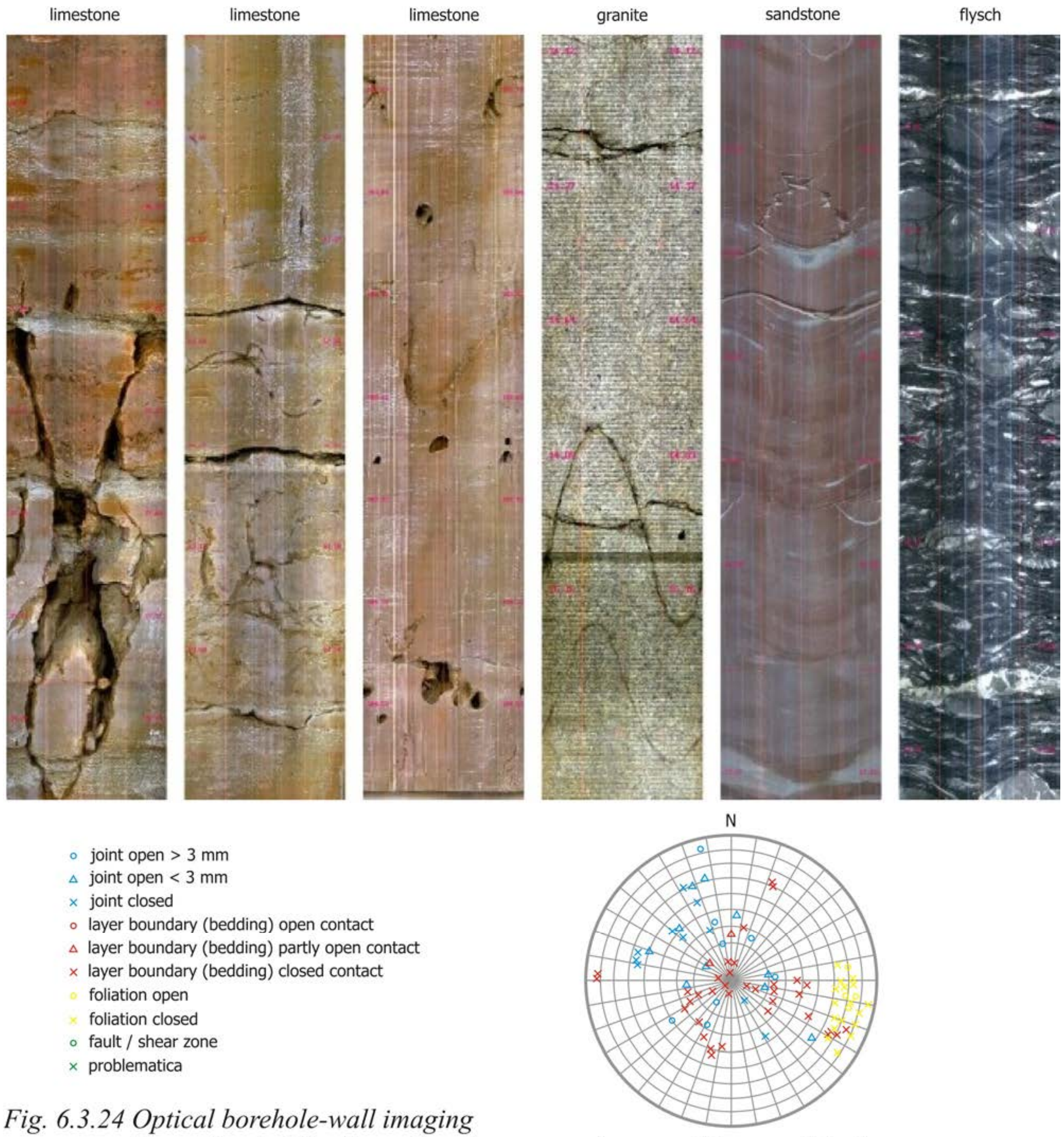


Fig. 6.3.24 Optical borehole-wall imaging (according to http://www.terratec-geoservices.com/struc opt.html)

difference between the sizes of the NNL and GGL anomalies. The largest NNL anomaly appears in the interval from 130 to 132 metres; a second occurs in the interval from 122.5 to 123.5 metres; and a third smaller anomaly in the interval from 137.5 to 140 metres. The GGL results show the largest anomaly in the upper fractured bed from 122.5 to 123.5 metres, with a smaller one at 130 to 132 metres and almost no detectable anomaly in the section from 137.5–140 metres. Sharp GGL anomalies always indicate that cavities are present, because an increase in the frequency of fractures does not significantly reduce the bulk density. The pattern of the NNL shows that the anomalies are caused by both cavities and joints in the granite, especially over the interval from 137.5 to 140 metres.

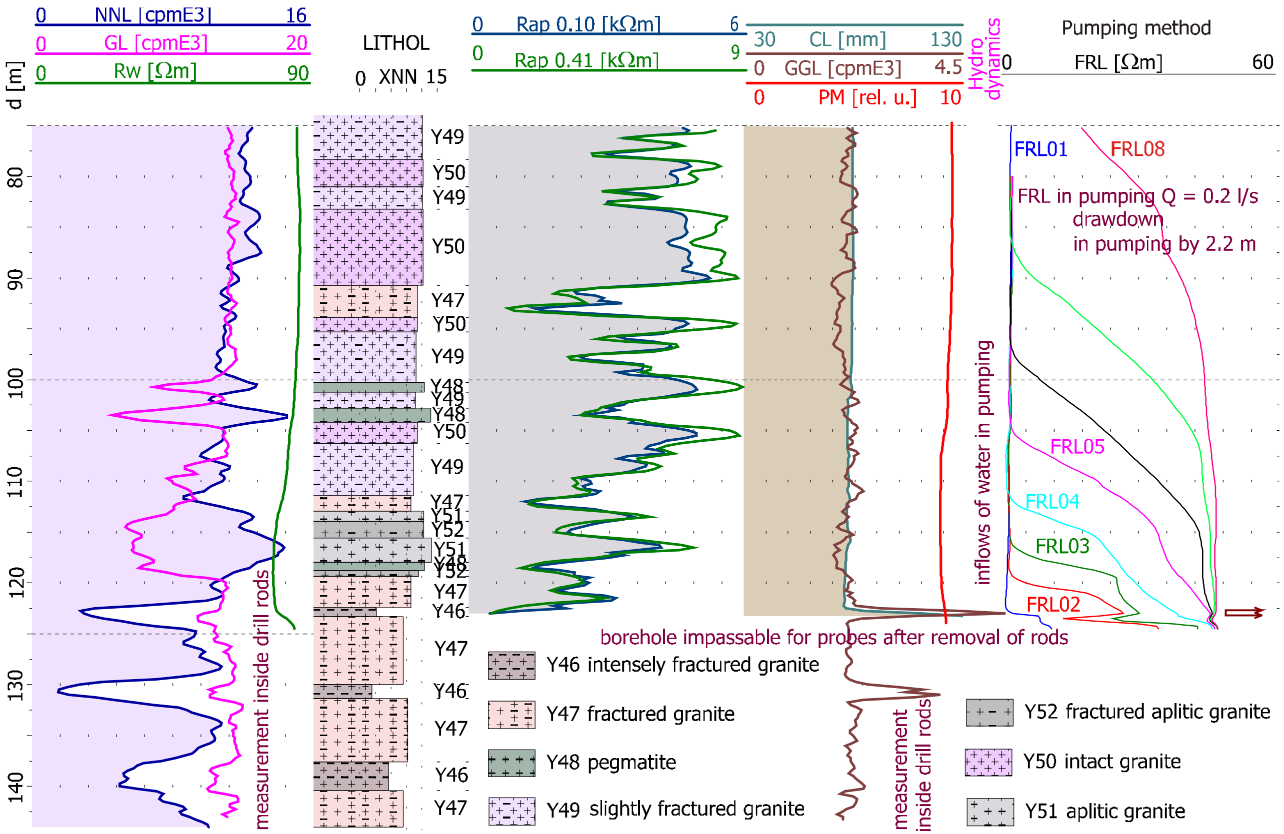


Fig. 6.3.25 Comprehensive logging using nuclear methods for measurement in drill rods (after Lukeš, 2006)

A progressive increase in knowledge about the geology at any site is essential so that objective decisions can be taken about the next stage of survey work in the area of interest. In some cases, circumstances are such that a conventional procedure for work flow is not possible, either for logging measurements or for other geophysical measurements made in the 'borehole – surface' mode. One of these circumstances is when it is necessary to case a borehole immediately after drilling is completed. In this case, the only option available is to carry out logging in the cased borehole or in the drill rods. Even in such cases it is possible to obtain information that will be helpful in answering particular questions about the geology and the physical properties of the rocks at the site.

An example of logging measurements carried out in perforated plastic casings in borehole JM1024 at a site in Brno is shown in Figure 6.3.26. The perforated sections of a string of casing can be identified readily by logging resistivity. The illustration shows a curve measured using a probe Rap of 0.41 metre in length. Solid casings and transitions between perforated casings are characterized by high values of apparent resistivity. When using sonic logging, it is possible to determine not only the velocity of longitudinal waves and the attenuation of seismic waves, but also to determine the mechanical properties of the rock directly by using modern software to process velocity parameters. In the example shown, the modulus of elasticity and the tensile strength of Neogene soils were determined using this procedure.

If a special study of the rock mass is required, it is easy to use portable rigs instead of the heavier installations normally used for logging purposes. In the 1970s, only devices that enabled measurements by a single method were available. Now, there are devices on the market which can make use of a range of different probes. The illustration in Figure 6.3.27 is of measurements made at the Dlouhé Stráně site. The aim was to assess variations in the weathering of the rock mass at the site planned for the construction of the lower reservoir of a PSHEP. The logging measurements were surprising because they showed that the bulk density in the near-surface layer was varying and that these changes could not be explained easily. If the superficial layer of debris is neglected, then the bulk density of the rocks in borehole HPV 7005 does not change with depth, but in some cases the bulk density decreases (borehole HPV 7002), and, in other cases, increases (borehole HPV 7004). Because a calibrated base was not available at the time the work was carried out, it is not possible to determine absolute values for the changes in bulk density. In this case, the variations appear to be due to a complicated combination of processes involving normal weathering in a humid climate and weathering processes under icy conditions.

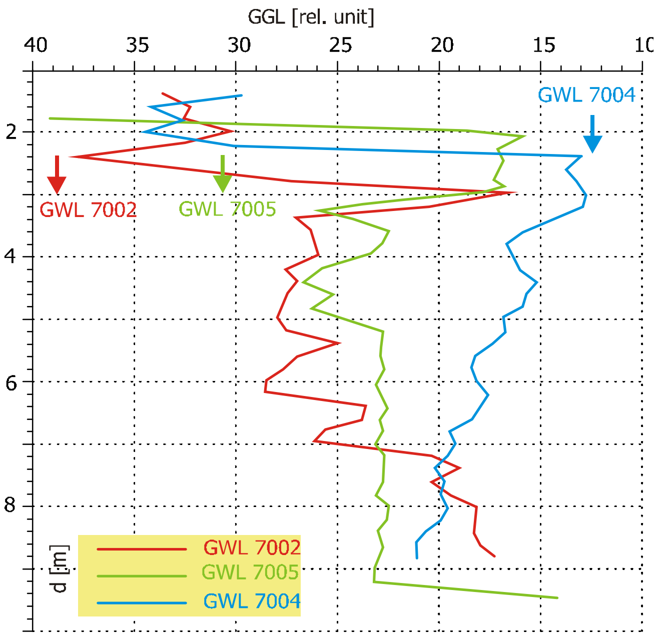
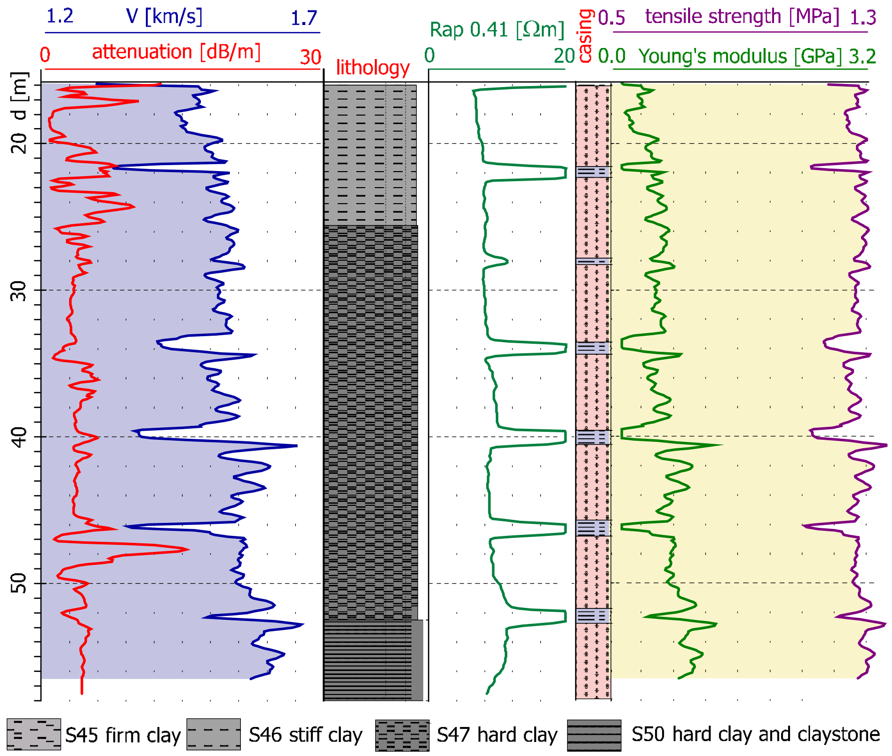


Fig. 6.3.27 GGL in shallow boreholes (adapted from Trávníček et al., 1971)



(after Lukeš, 2002)

Fig. 6.3.26 Logging measurements in perforated casings

Logging can provide a considerable amount of information about the hydrological regime at a particular site. An illustration of the processing and interpretation of such measurements is shown in the right-hand side of Figure 6.3.25, together with a description of the results. Recently, significant progress in hydrogeological logging methods has been made by the company Aquatest Praha, particularly in procedures to determine the direction of groundwater flow by measurements in a single borehole. Figure 6.3.28 shows the results of such measurements in one of the hydrogeological observation wells. The image depicted was not obtained by

North is at the top,
the flow direction has an azimuth of about 270°

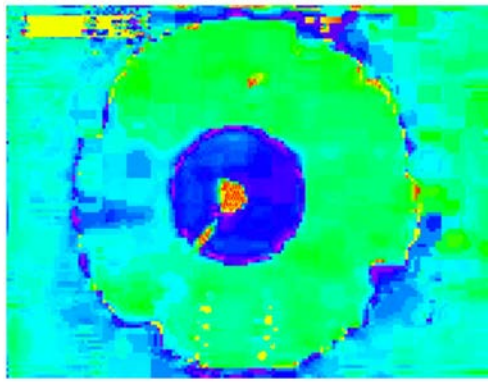


Fig. 6.3.28 Flow of water across a borehole (after Pitrák, 2008)

using a television camera but was created by digital processing of photometric measurements. Coloured liquid, which changes the transparency of water in the borehole, is spreading from the borehole in the form of a plume. By making repetitive photometric measurements, it is possible to determine the direction of groundwater flow. In this case the measurements show that water is flowing from east to west across the borehole.

Based on experience and from the analysis of the given examples, it is clear that a successful interpretation of the geological composition and structure of a dam site can be made only if different types of logging measurements are combined. To obtain the broadest possible picture of the geological makeup of a site, it is necessary to combine resistivity methods with measurements by nuclear logging, sonic logging, and additional measurements to monitor the technical state of a borehole. Moreover, it is appropriate to complement these measurements by hydrogeological logging methods and non-routine procedures, e.g. logs of magnetic susceptibility. To interpret the borehole logs correctly, it is necessary to make maximum use of all the geological and geotechnical information obtained from the borehole, i.e. core recovery, rock quality designation (RQD), maximum length of fragments, and/or measurements on the drill core obtained using a pocket penetrometer. Furthermore, a successful interpretation of the geophysical logs depends on a complete knowledge of the descriptive log of the borehole, not overlooking any information, even if it seems irrelevant at first sight.

Commonly, velocity curves show low rates of increase, and sometimes moderate jumps in velocity and, in certain cases, even a decrease of velocities with depth that is known as a velocity inversion. This fact must be borne in mind, especially when interpreting surface seismic measurements. It is necessary to take account of the fact that a pronounced seismic inversion can affect the interpretation of the depth of physical boundaries substantially. Therefore, in surface seismic measurements, it is necessary to pay great attention to the determination of velocities above the main refraction horizon. When possible, parametric measurements should be made so that the average velocity in the overlying bed can be determined using the results.

Conventional logging methods that are used in the survey of all dam sites include electrical measurements which are made in almost all boreholes. This is due to the simple design of the apparatus and the relatively low cost. Of the electrical measurements used for logging, resistivity is the most important. Almost every logging team is equipped with standard potential or gradient probes. It is natural that better results can be obtained by using laterologs, but short laterologs are not often available for use in shallow boreholes drilled for engineering-geological purposes. Sometimes interesting results can be obtained by using other electrical procedures, for instance measurements of spontaneous polarization. This does not often provide information about primary variations in the rock mass, but about changes induced by man, e.g. by cementation during drilling.

It should be noted that caliper logging is usually carried out to enable interpretation of the gamma gamma logs. This problem is avoided when using modern processing procedures in which, by calculating the effect of the borehole diameter, corrections are automatically applied to the GGL results which are given directly in units of bulk density. However, it is still necessary to pay attention to the shape of the caliper logs so that the scale of corrections which were applied when determining bulk densities can be estimated. Variations in bulk density changes caused by fracturing of the rock mass can be large, in some cases more than one gram per cubic centimetre. In these extreme cases, it is the presence of open cracks and not the fracturing of the rock mass itself that is responsible for the changes in density. Where a large volume of rock has been fractured, the decrease of bulk density is about 0.1 g/cm^3, but there are also cases in which this value is exceeded. Neutron logging can provide interesting information. Because this method is used especially to determine the clay content and the moisture in rocks, the processes that have taken place in sections near the surface and in fracture zones can be deduced. Gamma logging is another method of nuclear logging that provides valuable information about the lithological composition of the rock mass.

Useful information can also be obtained from logs of magnetic susceptibility. In many cases, it is possible to infer the degree of weathering of rocks based on the enrichment of iron oxides in weathered zones due to surface oxidation. This method can also be used successfully for checking the borehole casing. Logging methods used to monitor the groundwater regime at a site form a separate group. Information about the flow of groundwater could only be obtained with great difficulty by other means. Last, but not least, the possibilities of determining the distribution of stress in the vicinity of boreholes by logging should not be ignored. In this case, good results can be obtained by using a combination of geoacoustics and seismic radiography or seismic logging.

6.3.4 Other Geophysical Methods

Magnetic measurements are often carried out as part of the survey of a dam site. A prerequisite for their successful application is that there should be measurable differences between the magnetic susceptibilities of the types of rock that crop out beneath the dam site. When making magnetic measurements, some surprising results can be obtained. When interpreting measurements, it must be borne in mind that susceptibility is not the only parameter which affects the strength of the magnetic field at a given site. Another property of rocks which must be taken into consideration is remanent magnetization. The illustration in Figure 6.3.29 is from the Mohelno site where granulite and serpentine are the two main types of rock underlying the dam site. When planning geophysical work, it was assumed that the differences between the susceptibilities of these two types of rock would be sufficiently large to allow the successful application of magnetometric measurements. By the stage at which the first measurements were processed in the field, it turned out that the results were quite the opposite of what had been expected. The strength of the magnetic field over the serpentinite was smaller than that over the granulite. The only possible explanation was that there was inverse remanent magnetization of the serpentinite complex. This assumption was then confirmed by laboratory measurement using an astatic magnetometer. Despite this surprise, it was still possible to distinguish the two rocks from one another. This was particularly helpful in surveying the floodplain of the River Jihlava as shown in the figure.

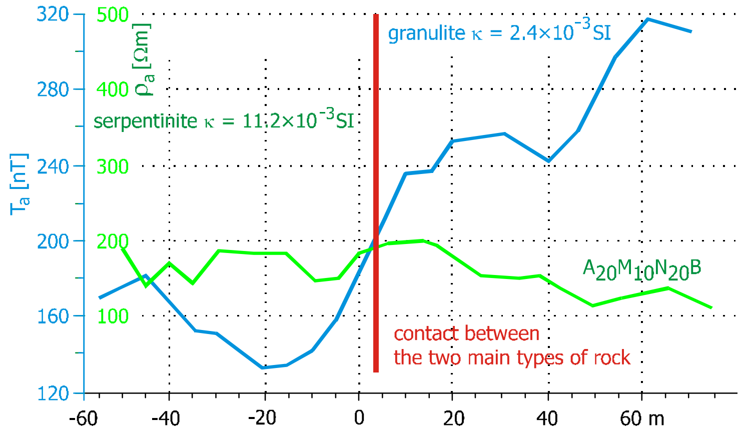


Fig. 6.3.29 Magnetometric survey at the Mohelno dam

Another illustration of magnetic measurements is from the Slezská Harta site. A profile at the edge of a basalt sheet was measured and the magnetometric measurements that were made can be used to illustrate the difference in the results produced by conventional magnetometric measurements and gradient measurement (Fig. 6.3.30). Field measurements showed that both methods for measuring the magnetic field gave almost the same results. Gradient measurement, however, has one significant advantage. During the field survey, it is not necessary to return to the base point. Thus, measurements made using this procedure are faster and therefore also cheaper. When measuring the magnetic gradient just above the ground, the micro-relief has an effect on the measured curves. Over the Culm outcrop it is also possible to detect the effects on the magnetic field produced by loose blocks of basalt and basalt debris. The best results are obtained from magnetometric measurements when the measuring probe is situated at a height of one to three metres above the ground surface. The figure illustrates two fundamental features. The first is the effect of remanent magnetization where the magnetic field is again lower over the intact basalts than over the Culm. When the basalts are fractured, the primary inverse remanent magnetization reduces and the magnetic field increases. Even under such conditions, however, the delineation of the basalt lava flow can be made more accurately by magnetic measurements than by resistivity measurements. Neither the narrow nor wide spacing of resistivity profiles provides a precise indication of the boundaries of the volcanic outcrop. In this case, resistivity is a better indicator of the fracturing and weathering of the rocks than of their lithological composition.

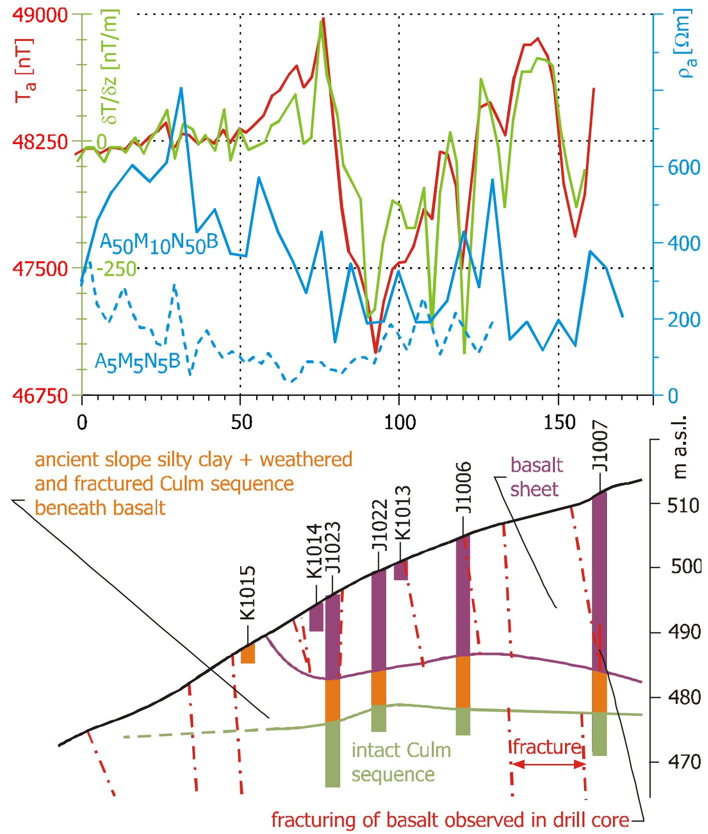


Fig. 6.3.30 Measurement of magnetometric gradient

When interpreting the geology of a dam site using the results of geophysical surveys, the information obtained from classical magnetometric work is not as great as that obtained by using geoelectrical, seismic and logging methods. Their value in helping to identify the lithological composition of a site is indisputable. In this case, the value of the method for locating buried blocks of magnetic volcanic rock has been demonstrated. Because measurements of the Earth’s magnetic field are simple to make, there will always be a case for carrying out a survey using magnetometry, even though the chances of a successful interpretation are sometimes low.

Gravimetric measurements are relatively little used in surveys of dam sites. Nevertheless, gravimetry can yield good results, especially when investigating the geological structure in the vicinity of high dams and, in contrast, when they are applied on a small scale in the search for cavities. The advantage of gravimetric measurements is they can be used for identifying zones in which material is absent, e.g., fracture zones, especially where open fracturing has been produced by tensile stresses. It is not only possible to identify zones in which tensile stresses prevail by using gravimetric measurements, but also to determine the depth of such structures and, sometimes, also to determine zones in which compressive stresses are concentrated.

When constructing high dams, it is absolutely essential to know the geological structure of the adjacent slopes. One of the possibilities is to carry out a survey using gravimetry. An example of a gravimetric survey made in the area of a slope failure at the Dolný vrch site is shown in Fig. 6.3.31. In this case it must be pointed out that the gravimetric survey was not carried out for the purpose of building a dam, but for a hydrogeological survey of the limestone structure of Dolný vrch, that extends across the border between the Slovak Republic and Hungary. Even though the stations used for measurements were spaced 200 metres apart, it was still possible to resolve the structure of the slope.

The first interpretation of the raw measurements was made by hydrogeologists and gravimetric specialists, but this was subject to a thorough re-evaluation. This showed that the original interpretation of fracturing on the slope corresponded neither to predictions about the failure of blocks on the slope, nor to the geology of the karst. To develop a rational explanation of the data, a model of the slope failure was created and a theoretical anomaly for it was calculated. The figure shows how the structure of the slope was interpreted using gravimetric measurements. The only boundary that cannot be determined reliably is the boundary between the intact limestone of the Middle Triassic and the underlying Werfenian sediments. The difference in bulk density between these rock types is only 0.01 g/cm^3. In such cases, it makes no sense to apply a gravimetric method.

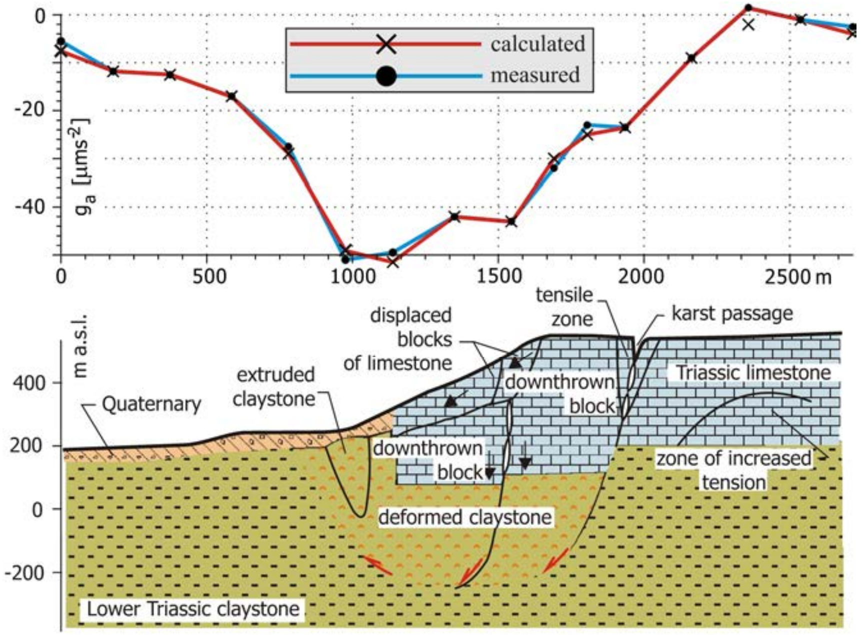


Fig. 6.3.31 Gravimetric profile across a steep slope in limestone (after Bláha, Mrlina, Nešvara, 1998)

If rocks are fractured, it is possible to determine the boundaries of fractured masses and their bulk densities by using gravimetric measurements. The most important result for an engineering geologist is that the depth of limestone blocks sunken into the Werfenian can be determined. Information about the gradual sinking of individual blocks is also valuable. It is possible to detect the depth to which remoulding of the Werfenian sediments has occurred. Plasticization reaches down to a depth of 350 metres. The deformation of the Werfenian rocks beneath the limestone also continues in the foreland area. The phenomenon whereby plastic rocks in the foreland of rigid blocks are deformed is described as “bulging”. In general, it can be stated that the remoulding of the Werfenian sediments reaches much greater depths than might be expected. The location of individual tensile zones with wedge or trapezium forms is another crucial piece of information provided by the gravimetric survey. For the engineering geologist and geotechnician it is important to know about the reduction of mass produced in zones of tensile stress and deformation. This data can be inserted directly into stability calculations. The increase in bulk density of a block at the base of the Triassic limestone is also interesting. Evidently, this has been caused by compression of a limestone bed as a result of flexure so that cracks that were originally open in this area are closing up.

The purpose of Figure 6.3.32 is to illustrate how zones of weakness where open spaces are present in the rock mass can be identified. The measurements were made in the Příbram area. In addition to the results of gravimetric measurements, the figure also depicts the results of seismic, geoelectrical and thermal measurements. Above the fractured mass at 60 to 90 metres along the profile, there is a distinct drop in the value of the acceleration due to gravity as well as in the velocity of longitudinal waves. The scale of the decrease in both these parameters indicates that the rock mass is not only fractured, but also that open spaces must exist in the section identified. The curve of the dipole electromagnetic profile (DEMP) and the temperature curve show no anomalous values that would indicate fracturing of the rock mass or the existence of open spaces.

The application of gravimetric methods to engineering-geological surveys has been increasing significantly in recent years. The reason is the ready availability of new microgal gravimeters. This type of instrument has enabled increases in the accuracy of gravimetric measurements by almost one order of magnitude. A second reason for the growth in use of gravimetric methods is the relative ease with which results can be processed on personal computers. It can be predicted that the application of gravimetric methods in surveys for dams will increase significantly in the future.

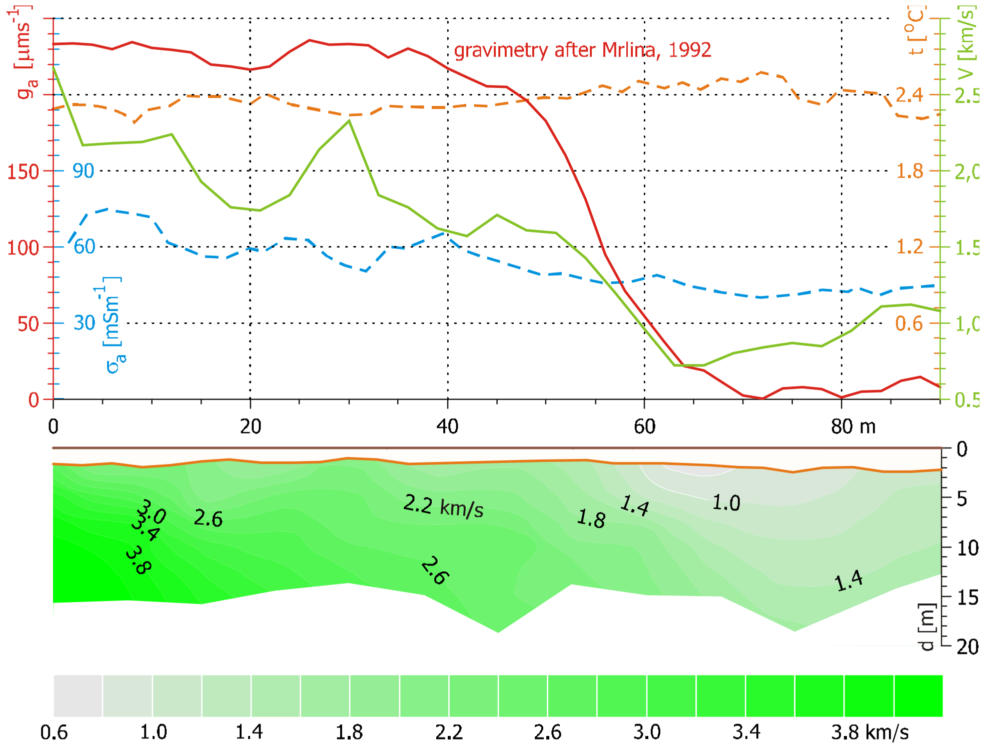


Fig. 6.3.32 Gravimetric measurement across a zone of weakness

Information about the behaviour of the rock mass can also be obtained by using thermometry. When contactless measurements of temperature are made, a large amount of data can be obtained relatively quickly and cheaply. The principle of the method makes it possible to use with success even in those areas where the temperature field of the Earth is unstable. Figure 6.3.33 is an example of measurements made in a tunnel at the Angat dam site. The tunnel transfers water from the basin of the River Umiray into a reservoir on the River Angat.

The area of this investigation lies in the southern part of the Sierra Madre Mountains. The basement is formed by metamorphosed rocks of Lower Cretaceous age and a mafic complex of Upper Cretaceous age. These geological units are covered by Upper Cretaceous, Palaeocene, Upper Eocene to Lower Oligocene, and Middle Miocene formations. The main magmatic activity during these periods was the intrusion of granodiorite. The Lower Eocene beds are deposited on the basement, partly conformably and partly unconformably. During the Upper Eocene to Lower Oligocene, intrusion of the granodiorite into older rocks took place.

Measurements were made in the tunnel during November 2007 and provided a whole range of interesting information. The results of all measurements are shown in Figure 6.3.33. One of the most important measurements in the tunnel was the measurement of the temperature of the lining. Measurements were made using an infra-red thermometer mounted on a moving truck. The differences detected were surprisingly large, ranging from 21.9 to 26.2 °C. The main anomalous zone extended from 5,500 metres to 8,200 metres. In this part of the tunnel, the temperature dropped by comparison with the normal field. If the undisturbed pattern of temperature in the tunnel is depicted by a red dotted curve, then the temperature anomaly in the middle of the tunnel amounts to almost three degrees. The explanation for such temperature changes must lie in the complicated pattern of groundwater flow in which juvenile waters may mix with vadose waters.

Fig. 6.3.33 Temperature measurements in the Macua - Umiray tunnel

The final example of geophysics used for the survey of dam sites is that of radiometric methods. The principle of the method means that it is used on rare occasions to survey the regime of tailings lagoons containing radioactive substances. In Figure 6.3.34, the results of the measurement of radioactivity in the vicinity of a wastewater treatment plant below a tailings lagoon are shown. In this case the content of uranium and the total α activity were monitored. Because the measurements were made repetitively over a significant period of time, it would not be possible to apply this procedure in the commercial survey of a dam site. Radiometric measurements should be one of the methods included in surveys carried out to monitor the operation of a working dam. In the present case, over a certain period of time, it was determined that the admissible limit of radiation had been exceeded. After adopting remedial measures, the radioactivity decreased below the prescribed limit (Poláček, 2000 – oral communication).

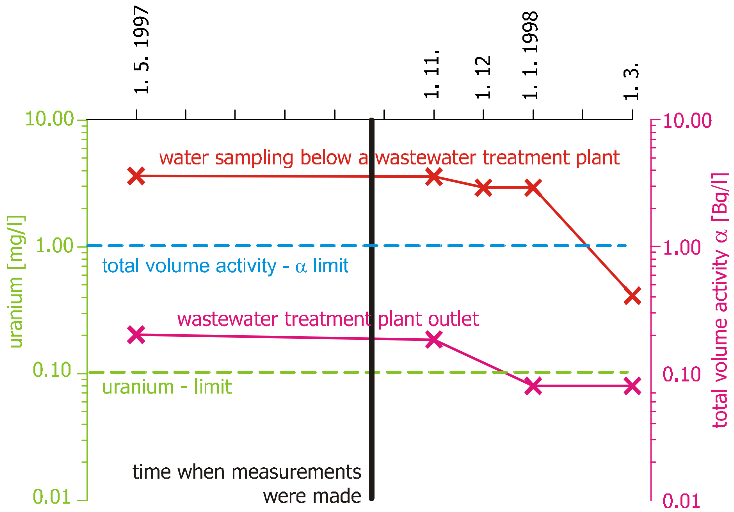


Fig. 6.3.34 Measurement of alpha tracks (after Poláček, 2000)

When geophysical measurements are carried out as part of the engineering-geological survey for a dam, the methods employed often enable savings in the cost of fieldwork. Experience over a long period has shown that this should not be the overriding consideration. Other factors in geophysical work are more important. The authors of this book are of the opinion that the most important advantage of geophysics is that it is possible to obtain information that would be unobtainable by other means, or when it can be obtained by other means, the costs in time and money are often prohibitive. There are many advantages in having direct access to new information gathered by the use of geophysics and this information can also be used to plan a strategy for the direct exploratory workings that will enable the geological composition and structure of the area of interest to be confirmed. Equally, this information will also enable the most suitable places for making other tests to be identified so that the geotechnical properties of the rock mass can be measured and the behaviour of the whole mass and its constituent parts can be predicted, especially in critical areas of the dam site. Only by taking such an approach can financial losses and catastrophes during the design, construction and subsequent operation of the dam be avoided.

7 Direct Survey Work

Without a detailed knowledge of the geological and hydrological environment, no dam or water-retaining structure can be built economically and safely. Despite all the progress that has been made in the development of new methods of geological surveying, direct investigation of the geology in the field remains the cornerstone on which the understanding of the geological composition and structure of an area rests. We use the term ‘direct survey work’ for operations in which rocks are exposed, either by excavation, tunnelling, mining, stripping, drilling, blasting or other direct procedures. Such investigations are fundamental for the successful engineering-geological survey of a dam site or any other hydrotechnical construction. New developments in equipment and technology have had an influence on the procedures used in direct survey work. In recent years more sophisticated drilling techniques that enable almost one hundred percent recovery of core, combined with logging methods and the optical documentation of boreholes, have been able to provide practically the same amount of information as exploratory tunnelling and shafts formerly did. A complementary procedure is also to replace the excavation of vertical shafts by triple boreholes which, in addition to the methods described above, enable geophysical measurements to be made using the ‘borehole – borehole’ mode. Such a procedure can provide an almost perfect three-dimensional picture of the geological structure.

7.1 Drilling Surveys

Drilling work carried out for engineering-geological and hydrogeological purposes differs in character from drilling carried out for exploration of mineral deposits and oil or natural gas fields. When drilling is planned on dam sites, the holes are relatively short, commonly not deeper than tens of metres and only exceptionally do boreholes reach depths of over a hundred metres. In such shallow boreholes, the proportion of unconsolidated Quaternary deposits (cohesive and loose soils) and weathered sections of the bedrock is very high but these are of particular interest in engineering-geological and hydrogeological surveys. This is in distinct contrast to the bedrocks intersected in deep boreholes drilled when exploring for mineral deposits. In the case of holes drilled for engineering-geological purposes it is necessary to recover the maximum amount of drilled material and to cause the minimum disturbance or change of state in the material during drilling. This requires the use of some very difficult techniques. Nevertheless, the procedures used for drilling must still meet the specifications demanded for geological and geotechnical investigations. For example, holes with specific and precisely defined diameters must be drilled so that the devices used to make engineering-geological measurements can be lowered into them. Holes drilled for hydrogeological purposes must allow for the casing and installation of equipment so that measuring instruments and pumps can be mounted. It is always necessary to choose the drill rig and drilling procedures that are compatible with the aims of the survey and suitable for the geological formations that are to be drilled.

In all cases the core must provide a complete and accurate record of the geological section. This depends primarily on the percentage recovery of the core. The essential requirement is that a prescribed high percentage of the core is recovered. This is one of the basic demands made by the geologist, and must be respected. To meet this requirement the technique used for drilling must be chosen carefully. Since the beginning of subsurface investigations of the geology of foundations, the maxim governing all work is that quality should be more important than quantity. Therefore, up until the 1960s, the typical equipment used for drilling was a simple windlass and a wooden tripod with a set of simple manual tools to enable dry drilling, using rotary, percussion or a combination of percussion and rotary methods. Holes drilled manually were usually 150 to 300 mm in diameter. If soil deposits were being drilled, for example when surveying the subsoil where clay was to be used in the cores of embankment dams, then 150 mm was chosen as the minimum borehole diameter to enable the collection of undisturbed soil samples by sampling instruments in capsules of 100 or 120 mm in diameter.

Using this hand-operated drill rig it was also possible to drill in coarse-grained gravel, even when some boulders were present. These were removed by the drilling team, often without the use of a heavy drill bit. However, this manual procedure was gradually replaced by high-performance machine-operated core rigs that operated using a flush of water or drilling mud. Rotary drilling is accomplished using a circular drill bit turning in the ground. It grinds rock only on the perimeter of the borehole, while the core of more or less undisturbed rock remains in the middle and as the drill bit advances this is progressively encased in a steel cylinder named the core barrel. During drilling, water or drilling mud is flushed into the borehole to carry away the mud and cuttings produced by the drill. The core barrel itself is usually two to three metres long. In softer and fractured rocks, a double core barrel is commonly used; the inner barrel serves to protect the core from flowing water, cuttings and mud.

In drilling, the recovery of core is the factor that determines the success of the survey and drill bits of diverse patterns are used. The choice of drill bit depends, above all, on the character of the rock to be drilled. The drill bits are divided according to their structure into saw tooth, button, shot and diamond drill bits. In solid rocks with a minimum of discontinuities, diamond bits are used to obtain long smooth sections of core. Diamond bits however cannot always be used. Other types of drill bits, therefore, still form part of the arsenal of drilling technology.

Recently, new trends in drilling technology are beginning to gain ground. Rigs with a wire-line system are being used more and more. In this procedure the core barrel is suspended on a cable that passes down the drill string. Core recovery is very fast because it is not necessary to pull the whole drill string. After the inner barrel is filled with core, the barrel is retrieved using the cable inside the drill string. One advantage of this technique is that the drill string separates the core barrel from the borehole wall during recovery and the core is thus largely protected from drilling fluid. With this drilling system, double core barrels can also be used in tectonically fractured zones. In combination with flow-through diamond drill bits, the core is protected to the maximum extent. The use of this technology for engineering-geological purposes is more or less essential because of the high core recovery that is demanded. In special cases, throughout the world, a technique is beginning to be used in which the core is continuously protected by a polyethylene sleeve during the drilling process. Both the drilling techniques described above have enabled marked improvements in core recovery and, thanks to these procedures, it is not unusual that even clay gouge can be successfully recovered intact from fault zones.

For sections of exploratory boreholes in the bedrock, including boreholes for testing water pressure and making injection tests, and/or for pressiometric tests and logging measurements, a diameter from 76 to 150 mm is commonly used. Core drilling provides perfect samples, from which not only the character of rocks but also the dip of bedding planes or joints can be determined. Based on the character and frequency of jointing, the technical properties of rocks and their workability can be assessed. Even by using a strictly optimized traditional procedure for drilling, only in very exceptional cases can a core recovery of 100 % be obtained. For example, in the thickly bedded limestone at the Centro Cuba site, the core recovery reached 80 to 90 %, while in thinly bedded shaly rocks the recovery was only 60 to 70 %. This is why logging measurements in boreholes prove so useful. They not only enable geological descriptions to be made, but also provide information about the physical-mechanical parameters of the rocks intersected by the borehole.

In addition to static rigs, mobile rigs can also be used successfully for engineering-geological investigations. It is possible to manoeuvre skid-mounted rigs even onto high slopes. Today, the use of rotary drilling machines with a range of efficient drill bits and wire-line technology that are capable of providing high core recoveries and intact samples for laboratory analyses is becoming commonplace in engineering-geological surveys of dam sites.

Hydrogeological wells, if they are to be used for pumping tests, must be drilled with diameters wide enough to allow for the casing needed so that submersible pumps can be lowered into them. A drill diameter of at least 150 mm or larger is usually chosen. If the borehole is used to investigate the composition of the soils on a site, then a borehole diameter of at least 200 mm is chosen so that undisturbed samples of soil can be collected

In order for a drilling survey to produce satisfactory results, precise records of work flow must be systematically maintained. Records of work flow, as well as preliminary descriptions of rocks and core recovery are mostly made by the drilling foreman or a technician and must be checked regularly by a professional geologist. The drilling foreman and the drilling technicians are responsible for ensuring that the optimal drilling procedures are being used and that the best possible recovery of core is being maintained. It is not reasonable to expect that they should also carry out the work of the geologist. In cases of dispute, when a drill core is lost or even when lost drill core has been substituted to ensure that the recovery matches the prescribed standard, or simply when the core has been wrongly described, it is appropriate to make measurements with logging instruments to check the results.

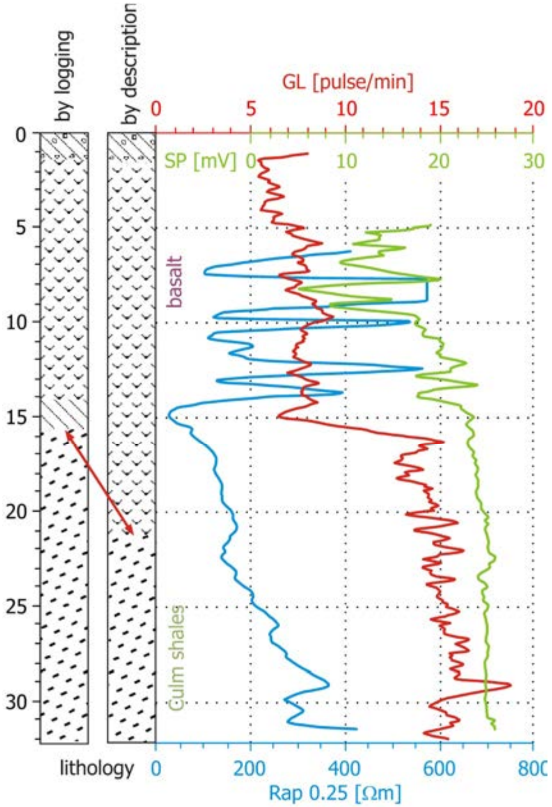


Fig. 7.1.1 Error in lithological description of core detected by logging

In Figure 7.1.1, an illustration of a "false" description of a drill core is given. An exploratory borehole was sited so that it would pass through a layer of basalt and into the underlying rocks of the Culm. The geological description of the suspect borehole is given in the right-hand column in Figure 7.1.1. The results of logging measurements, however, suggest that the basalts are only 14 metres thick instead of the 21 metres stated and, based on the results of resistivity logging, it is possible to identify a bed of silty clay more than one metre thick buried beneath them. The Culm sequence begins at a depth of 15.5 metres. If a probe for making magnetic susceptibility measurements had existed at the

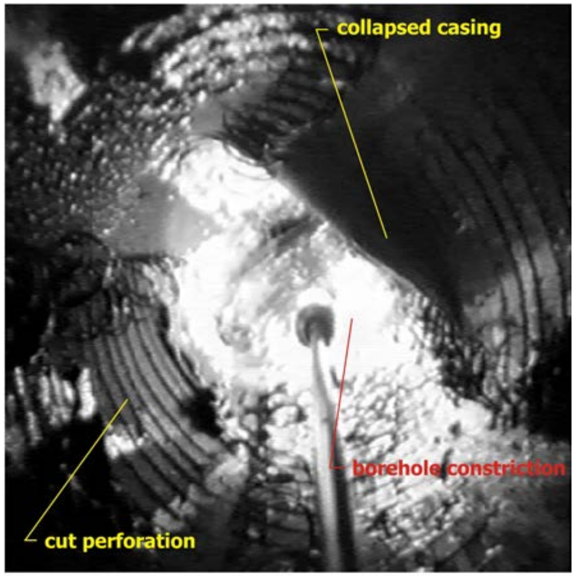
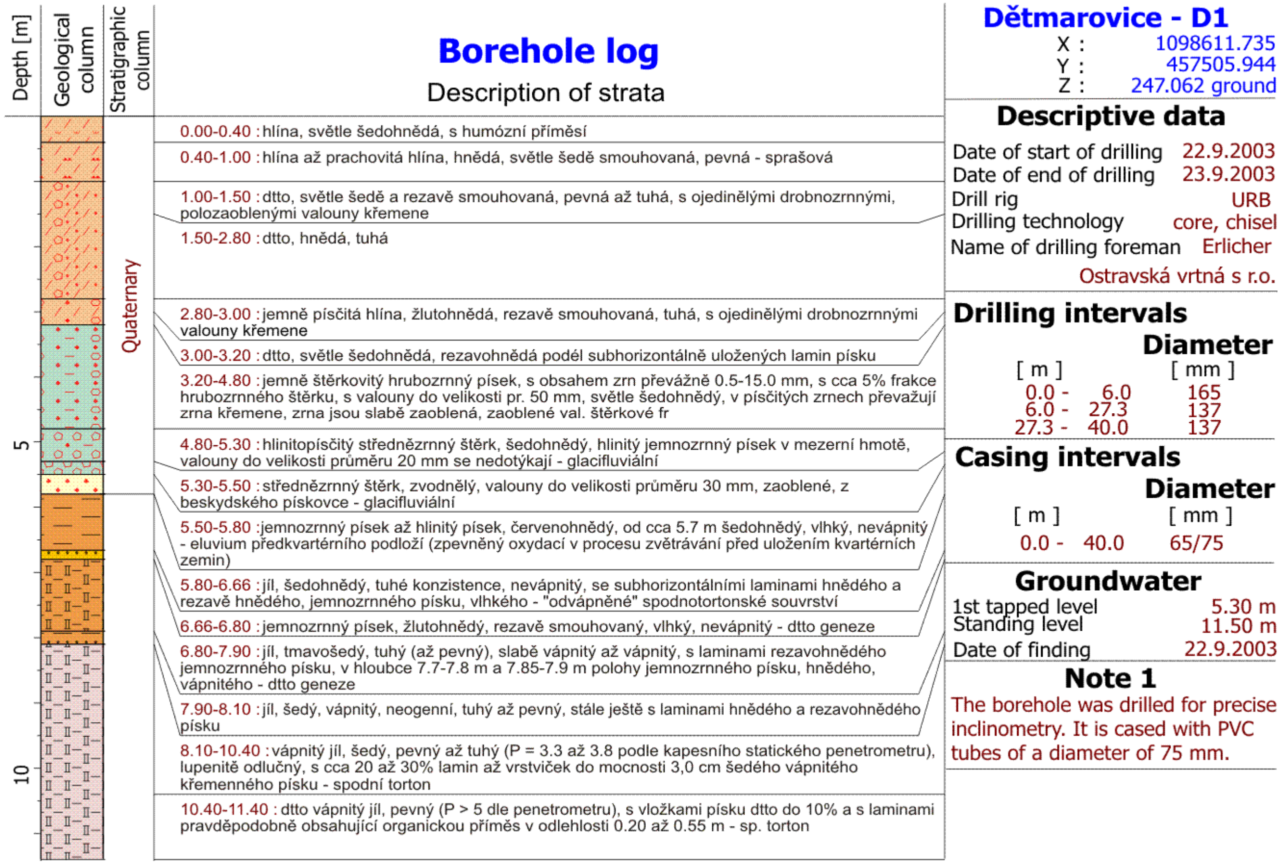


Fig. 7.1.2 Damage to borehole casing

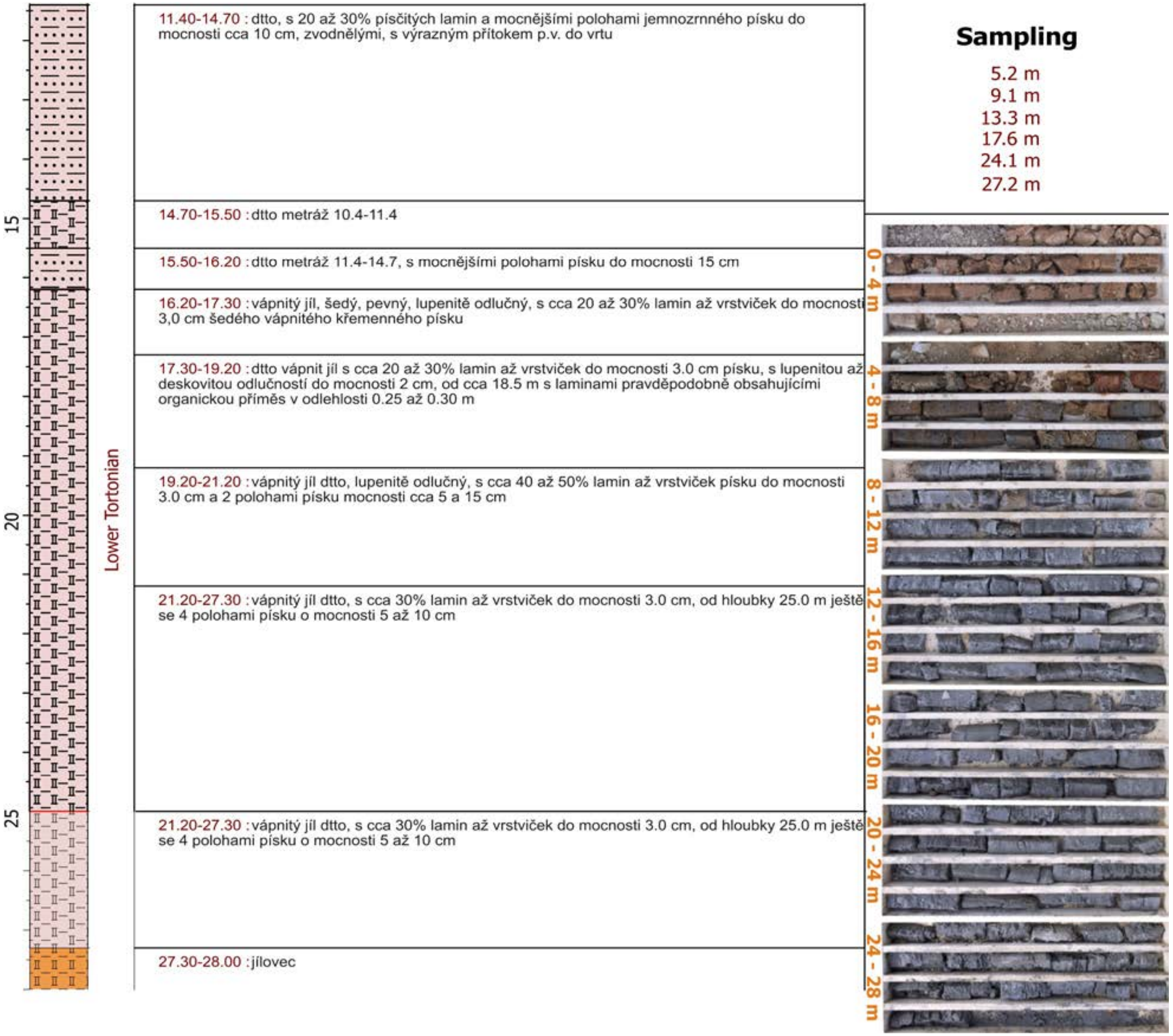
time of the survey, the geophysical definition of the boundaries would have been still clearer. Also, it was not technically possible to inspect the borehole using a television camera at that time. There were also errors in reporting the diameters of the borehole because the diameters recorded were larger than the real ones. The character of the walls of the borehole and the local lithology excluded the possibility that the drilled rocks had swelled.

In some case, problems are detected after the borehole is completed. Frequently a smaller depth than the drilled depth is lined, or the passage through a casing string by a measuring device is restricted. In such cases, several questions must be asked. Is the borehole usable for the designed purpose? Can the borehole be somehow repaired? Will it be necessary to drill a new borehole? How large is the additional cost of solving the problem? In all cases it will be necessary to assess the actual state of the borehole. A suitable procedure is to inspect the borehole using a television camera. As a rule, this will clearly show the reason why the diameter of a borehole has decreased or why it is impassable. The slender construction of the cameras used today sometimes even allows the camera to pass through a damaged section of a borehole. The illustration in Figure 7.1.2 is from an inspection carried out in an observation borehole using a camera producing black-and-white images which was not able to pass along the drilled length. It turned out that a casing string had been inserted using too much pressure and as a result it had collapsed where it had been weakened by the perforations cut in it. As a result the casing collapsed at this point, making the borehole impassable at a greater depth.

Fig. 7.1.3 Comprehensive graphic borehole log



An essential part of any drilling programme undertaken during the engineering-geological investigation of a site is the comprehensive description of the drill core. As in the case of other survey procedures, the rule is that the maximum amount of information about the rock environment and its behaviour should be obtained from any exploratory working. Although certain information may appear to be in excess of that required for a given stage of a survey, long experience has shown that the investment of time and effort put into the full and precise description of exploratory workings will always pay off.



An example of a graphic log of a drill core is given in Figure 7.1.3. This illustrates the system that is routinely used in the Czech Republic at the present time (the original description of the drill core in Czech is shown in this figure). This method of logging records the basic written description of the core as well as a graphic depiction of important characteristics, together with information explaining the purpose of the borehole, its location and coordinates, the time taken and technique used for drilling, the drilling tools employed, the borehole casing, the groundwater levels encountered and the standing water level. In addition, the intervals from which samples have been collected for laboratory tests are marked, as well as the method used for casing the borehole and unusual features observed during drilling. It is appropriate to complete this set of data with photographic images of the drill core.

Penetration tests can also be considered as a special type of drilling work. The procedure used for penetration tests involves the insertion of specially designed tools into soils so that the resistance of soils to this action can be measured. The devices used to carry out the penetration test have conical tips that are driven into the soil. When the conical tip is driven into the soil mechanically using a slide hammer, the penetration test is described as dynamic (SPT), and when it is pushed steadily, the test is described as static (CPT). Dynamic penetration testing is suitable for sandy and cohesive soils containing small gravel. The depth that can be reached depends on the device used; it is stated that it is possible to reach a depth of 30 m. Static penetration is applicable to loose and medium-dense sandy as well as cohesive soils; it can reach a depth of 100 m.

7.2 Mining Work

Mining and stripping, especially the excavation of pits and trenches for the purpose of determining the thickness and character of the superficial cover, the depth to the pre-Quaternary basement and to make measurements of structures in the rock mass, is carried out especially in areas where there is little or no exposure. These excavations are used both for the purpose of engineering-geological mapping of an area designated for the construction of a dam or related structure and so that the engineering-geological conditions governing the construction of the dam can be specified. Pits with a cross-section of 1.8 × 1.2 metres are excavated if the depth does not exceed eight metres and if their purpose is chiefly to enable a description of the geological section exposed in them. The depth of pits is chosen according to the technical requirements of the survey. As a rule, the depth will not be greater than 20 metres but, if necessary, it is possible to sink shafts even to larger depths.

A deeper section exposed by digging a pit is especially necessary when it is essential to know the geological conditions at depth or because it is necessary to carry out geotechnical tests in the field or for other specific reasons. In such cases, a cross-section of 2 × 15 metres is usually chosen, but the dimensions can also be larger. The reason for this is that mining safety regulations require a ladder compartment for access and a separate extraction compartment and, where inflow of groundwater is anticipated, allowances must be made for installing a pump in the pit. An engineering-geological survey of the sites for the extraction of deposits to be used in the construction of an embankment dam on the River Lomná in the Beskydy Mountains can be used as an example of these procedures. Pits were excavated down to a depth of 15 to 20 metres in the upper gravel terraces (1963). Because of the behaviour of the gravel in the terraces, it was only possible to leave short sections excavated to a depth of one metre for a few hours before it was necessary to install horizontal shuttering to prevent the working from caving in. This placed great demands both on those responsible for excavating the pits and on those responsible for describing the exposed sections because the process of excavation, description and shuttering was more or less continuous. It should be noted that, in the 1960s, indirect methods of observation using geophysical methods that would have made the almost continuous shuttering of pit walls unnecessary, were not yet available in the former Czechoslovakia. If pits are excavated in water-bearing rocks, the direct observation of the geological features exposed in the pit is still more difficult because the walls must be fully shuttered almost immediately. For this reason visual inspection *in situ* is not practical and reliance must be placed on samples taken during the process of excavation. Today, this disadvantage has been overcome by applying geophysical methods of documentation. These will be described in Chapter 7.4.

Excavations are more suitable than boreholes for surveying deposits of material that have a complicated history of deposition, because they enable the direct description of exposures in their walls and sampling can be carried out to provide a fuller range of more objective information than can be obtained from a borehole. Trenches and terrace cuttings are also made for the same purposes and machinery can be used to make them. The width chosen for a trench is usually from 0.8 to 1.2 metres, because wider trenches are very difficult to support and the amount of excavated material is obviously greater. In cohesive soils or in partly consolidated sand and gravel, trenches can be excavated to a depth of around four metres. The walls remain stable only for a very short time and it is therefore necessary to make geological descriptions and take photographs immediately, as well as to collect samples for subsequent laboratory investigation.

An example of the use of a cutting 125 metres long to verify the geological interpretation of a dam site is that made in the foot of the right slope at Dalešice (1972). This cutting confirmed the trend of the main fracture zone ("mylgramph") separating amphibolite from granulite and enabled the distribution of the separate types of rock to be delineated and the attitudes of other systems of fractures to be defined. In addition, the cutting enabled a geotechnical classification of the site into individual quasi-homogeneous blocks (Fig. 3.3.7).

Exploratory tunnels are commonly used for surveys of dam profiles or to identify sites suitable for underground caverns in which a powerhouse can be sited. In order to prepare sites for testing rock mechanics or for understanding a geological structure, it is possible to drive blind cross-cuts from a tunnel. The excavation of tunnels is very expensive and for this reason they are mostly used for dams with a height greater than 30 metres, and especially in cases where the geological conditions are complicated. At Dalešice on the River Jihlava, tunnelling was extensively used to make the engineering-geological survey for the construction of an arch dam. In this case a very thorough investigation was necessary to assess the condition of the rock mass, particularly at the places where the wings of the dam were to be keyed so that the stability of the dam during construction and after commissioning could be guaranteed. In all, thirteen exploratory tunnels with a total length of 737.5 metres were driven. Two other exploratory tunnels with a total length of 131.7 metres were later driven after the project was changed to the construction of an embankment dam. Sometimes, a survey by tunnelling is carried out where the tunnels will be used subsequently to divert water during the construction of a dam, or where they may form part of the permanent hydraulic system. The cross-section of exploratory tunnels is at least 1.2 × 1.8 metres. When excavated material is transported along a rail track in longer tunnels, it is necessary to make the tunnel at least 1.5 metres wide and at least 2.2 metres high. The exploratory tunnels A and B at the Centro Cuba site were excavated with these dimensions (Fig. 7.2.1). They reached a total length of 255.3 metres and 150 metres, respectively. In both the tunnels, several blind cross-cuts were made in which expansion loading tests and shear tests were made on blocks of rock.

Fig. 7.2.1 Mouth of tunnel G183, Centro Cuba (1984) (a photo by O. Horský - 1984)

Exploratory tunnels are very useful to establish the depth to which pre-Quaternary rocks are loosened by weathering so that they can be removed during construction. It is necessary to avoid driving exploratory tunnels parallel to the valley below the bottom level of the footing because, even if they are filled in subsequently, they could become preferred pathways for seepage after the water-retaining structure is commissioned. Even when driving exploratory tunnels in directions transverse or oblique to the slope of the valley, it is essential to proceed carefully to avoid breaking up the bedrock at outcrop, and to avoid overbreaks or caving in the tunnel. It is also important not to cause collapses that reach the surface of the slope. If field tests are to be made in some sections of a tunnel or in blind cross-cuts, it is necessary to break up the rock without using explosives so that unnecessary fracturing of the rock is prevented and the original stress in the surrounding mass is not catastrophically relieved. When investigating a route for the excavation of a diversion tunnel, or when tunnelling for other purposes, pilot tunnels are driven before the construction of the main tunnel which will be of larger dimensions adapted to the future method of tunnelling.

7.3 Scope of Subsurface Exploration Work

Drilling, mining and stripping work is carried out to the extent necessary to meet the objectives of the successive stages of a survey from the initial design to the subsequent engineering-geological stages that arise from it. At a dam site, these excavations must enable the construction of an engineering-geological profile across the axis of the dam and the main facilities so that the overall geological composition and structure in the area of construction of the dam can be properly revealed and understood. It must be borne in mind that it is especially important to identify and investigate all heterogeneities in the rock mass (e.g., faults, fractures and joint systems) by direct exploration. The depth and topography of the erosion surface must also be established, as well as the composition and physical properties of the soils and rocks and the location of the important geological boundaries. It is also important to find out the depth to which the rocks are weathered and the nature and shape of the weathering profile, together with the thickness and character of the Quaternary cover. The physical-mechanical properties of individual types of rocks and soils are determined by observation and by making field tests directly in exploratory workings; samples collected from the workings are also used for further tests in the laboratory. Exploratory workings are also used to verify the depth of the groundwater level and to establish hydrogeological parameters by conducting pumping and slug tests, water pressure and injection tests, etc.

It is clear that without the benefit of the exposures created by exploratory workings it would be impossible to make the necessary observations, field tests and measurements needed to understand the composition and structure of the rock mass and to establish the geotechnical and hydrogeological parameters that are required for engineering purposes. Without this information, decisions would have to be based on hypothesis and speculation. Survey work carried out directly in excavations and boreholes is always planned in relation to the knowledge gained earlier by indirect methods so that the survey can be made as efficient and cost-effective as possible.

At the stage of the orientation survey, drilling may be carried out along several alternative dam profiles. Boreholes, and/or other exploratory workings, are located along the most promising profile for the construction of the dam to gain a preliminary idea about the suitability of the site. Where a proposed site has a complicated geological structure or where there are hydrogeological problems caused by karst phenomena or other physical features that are likely to threaten the construction of a dam, or make it completely impossible, it will be necessary to extend the scope of the survey by direct exploratory workings at an early stage so that the extent of the structural and hydrological problems can be clarified.

In rocks with high permeability, it is necessary to locate boreholes to intersect the groundwater table on the slopes at an elevation at least as high as the predicted maximum level of the predicted maximum water level in the planned reservoir. Borehole J12 in the right bank of the Slezská Harta dam profile is given as an example. In this case, a hole was located at a distance of 250 metres from the keying of the dam on the right bank to intersect the groundwater table at a level corresponding to the future maximum reservoir water level (Fig. 7.3.1). It was necessary to case this distant borehole so that it would be possible to carry out long-term monitoring of the groundwater table.

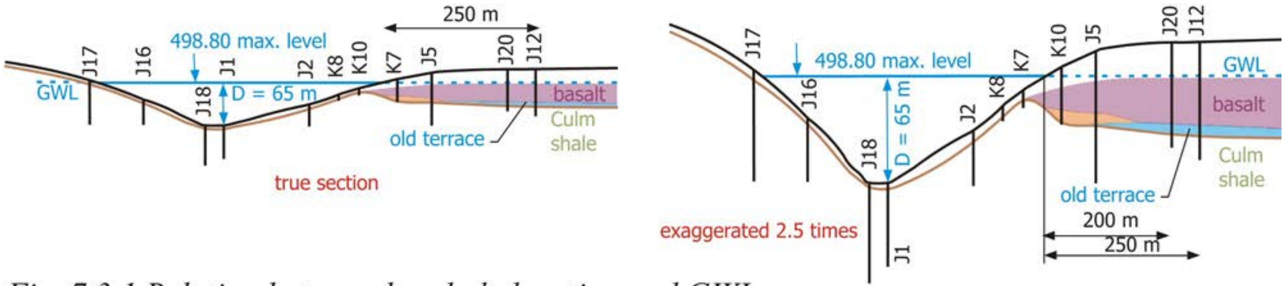


Fig. 7.3.1 Relation between borehole location and GWL

In certain cases, where there are rocks of extremely high permeability and the presence of a layer of low permeability in the subsoil offers a possible solution, it will be necessary to trace the position of the relatively impermeable bed during the preliminary stage of the survey. An example is the engineering-geological survey for the Charco Redondo dam in the Sierra Maestra Mountains where a unit of impermeable tuffitic shale was located beneath karstified, highly permeable limestone. This tuffitic shale forms a buried ridge in the right bank of the dam, the height of which rises towards the future reservoir level of the reservoir (Fig. 7.3.2). It became a priority to verify the shape of the ridge of impermeable tuffitic shale by drilling so that the dam profile could be sited where this ridge reached a level at least equal to that of the maximum reservoir level.

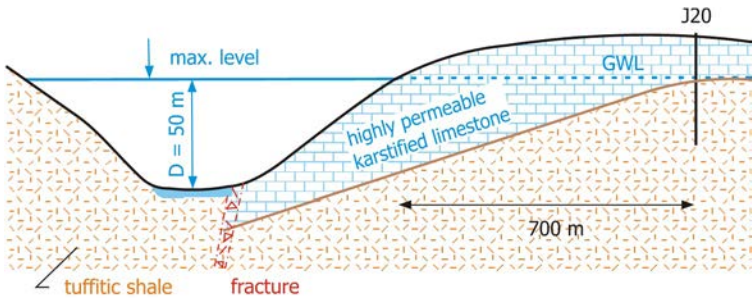


Fig. 7.3.2 Relation between location of borehole and impermeable basement

Where rocks or soils at a surveyed dam site are highly permeable, intensely karstified or subject to swelling, collapse or slope failures, it is necessary to carry out a special engineering-geological survey to assess the effect of these phenomena on the construction of the dam and associated facilities. In order to obtain detailed information about the likely problems, it is first necessary to carry out a geophysical survey so that a rational plan for direct exploratory workings can be prepared. At the preliminary stage of a survey, it is still possible to decide on the maximum spacing between exploratory workings based on experience acquired in previous surveys of dam sites. In areas where engineering-geological conditions are simple (A), the spacing will be from 250 to 400 metres, in the category of medium complexity (B), the spacing will be from 200 to 300 metres, and in the category of complicated engineering-geological conditions (C), the spacing will be from

150 to 250 metres. These are only tentative recommendations because exploratory workings must be located so that all of the main features of topography, geological composition and structure in the valley selected for the dam site can be accurately defined.

If, based on the results of engineering-geological mapping or a geophysical survey, substantial changes in geological or hydrogeological conditions or stability of slopes upstream or downstream from a dam are predicted, these problems must be addressed at the preliminary stage of the survey. As explained above, the peripheral exploratory workings must be located so that unweathered or stable rocks above the elevation of the maximum reservoir level can be intersected and, where the rocks are very permeable, the peripheral boreholes must be sited to intersect the groundwater level in a section higher than the predicted elevation of the reservoir. If seepage can occur through a wide near-shore zone, it is necessary to know geological and hydrogeological conditions beyond the horizontal contour of the maximum reservoir water level. Preliminary calculations can be based on the pattern of depression curves on the valley slopes. It must be borne in mind however that, especially in deep valleys in mountain regions where the rate of infiltration of precipitated water is low and runoff on steep slopes is fast, the depression curve in slightly permeable rocks may be very flat.

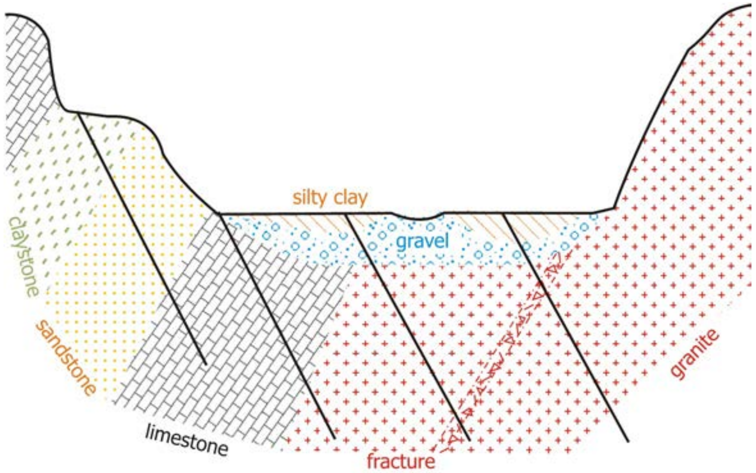


Fig. 7.3.3 Pattern of boreholes across steep geological contacts

Survey work must be planned in such a way that the important stratigraphic contacts (faults, tectonic fractures, unconformities, etc.) in a profile can be intersected. If the dip of beds across a dam profile is steep or if there are upright fold structures and steeply inclined faults, it will be necessary to excavate exploratory tunnels, or to drill inclined or horizontal boreholes (Fig. 7.3.3). If a conventional borehole cannot be drilled in a river bed, an inclined borehole can be used to locate the base of erosion. In cases where boundaries are obliquely inclined, the results of geophysical measurements are difficult to interpret. In the case of geoelectrical and seismic measurements, the results are most difficult to interpret when boundaries dip at an angle of about 45 °.

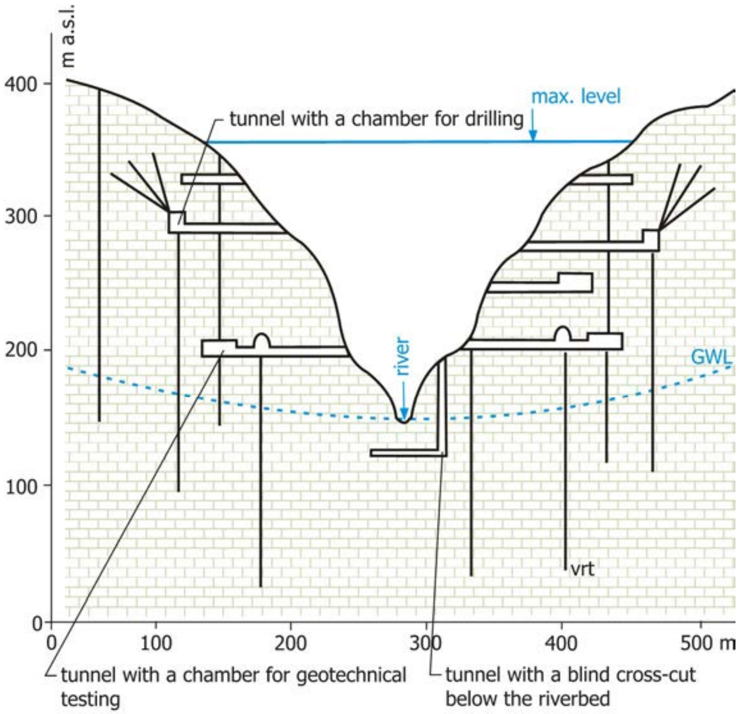


Fig. 7.3.4 Layout of boreholes and tunnels in a mountain valley

The type of exploratory workings chosen for a survey will depend on the geological structure and relief of the site and on the objective of the exploratory workings (e.g., a tunnel excavated to enable rock mechanics tests). In flat, wide valleys with moderate terrace gradients and thick Quaternary deposits, boreholes are most often used to determine the geological structure. If it is necessary to utilize the gravel or other material from a higher terrace for building purposes (for the construction of the dam, etc.), pits are more suitable. For example, at the Dolní Lomná dam site, pits down to a depth of 15–20 metres were used to confirm the thickness of gravel accumulations.

In valleys in mountain areas, a series of exploratory tunnels at different levels are the most effective way of obtaining information about the geological and tectonic conditions. Holes can also be drilled directly from blind cross-cuts underground and vertical raises can be driven from them to intersect important geological features (Fig. 7.3.4). The use of tunnels is particularly important when carrying out a survey for an arch dam so that the zones of stress release in the rock mass and the main zones of weakness can be identified. Load and shear tests can be planned in relation to the observations made in the tunnels and enable the mass to be classified into quasi-homogeneous blocks. In the engineering-geological survey for an arch dam at Dalešice, a total of 13 exploratory tunnels with a total length of 737.5 metres were excavated, 125 boreholes with a total length of 5,940.8 metres were drilled and 35 pits and trenches with a total length of 428.7 metres were dug (Fig. 7.3.5).

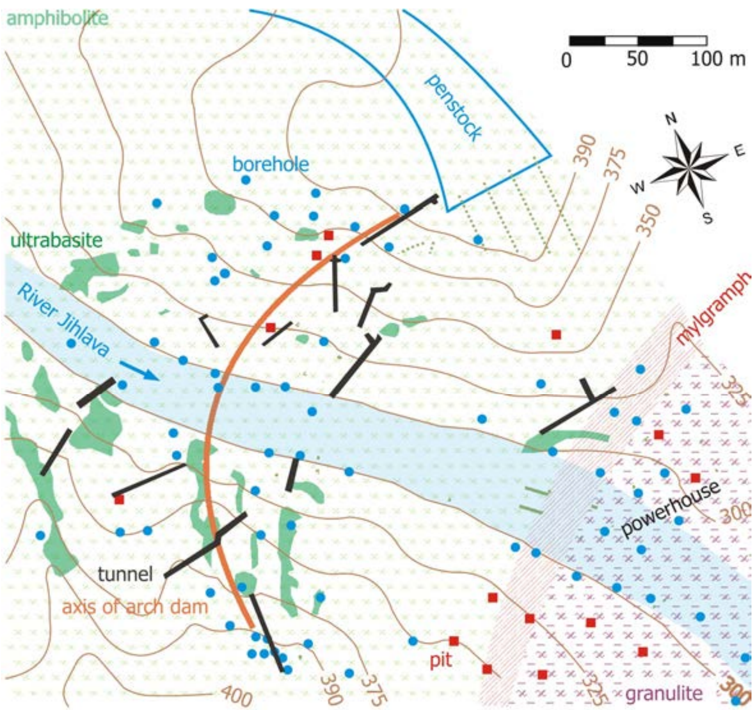


Fig. 7.3.5 Layout of exploratory workings at the Dalešice dam

To determine the thickness of Quaternary sediments in a valley floor or on slopes, it is necessary to end the holes several metres into the underlying bedrock to avoid the possibility that the hole has intersected a boulder or a loose block lying above the true bedrock. This was the problem during an engineering-geological survey of the Brumovice dam profile where drilling was carried out using a manual rig (1961). Large boulders encountered in glaciolacustrine sand and gravel created the illusion that the solid Culm basement had been intersected. Errors of this type are not uncommon and can have serious effects on the design and construction of a dam and its related facilities. On sites with the deeply and irregularly weathered pre-Quaternary basement or in areas of intense tectonic fracturing, the same sort of problem can arise. For example, in an engineering-geological survey for the dam at Slezská Harta, a series of shallow pits were dug exposing relatively sound Culm shale overlain by Quaternary sediments. Deeper pits and boreholes, however, exposed rocks that were intensely fractured and weathered to the extent that even silty clay with debris had been formed. These examples demonstrate that all survey work must be carried to a depth at which there can be no doubt about the position of important geological contacts and structures and the strength and stability of the rocks on which the dam foundations will rest.

At the preliminary and more detailed stages of a survey, profiles along the planned axis of the dam and parallel profiles on the upstream and downstream sides of the dam are investigated. The spacing between these profiles and the total number will depend on the width of foundations and the height of the dam, the dimensions and depth of the required excavation, and the complexity of geological and hydrogeological conditions. A detailed survey should be carried out along at least three profiles, one along the dam axis itself and the other two on parallel lines spaced about 50–100 metres apart. All survey work must be planned so that the results provide information allowing the compilation of geological sections in directions along cross-sections parallel and perpendicular to the profile of the planned dam and at sites where other important facilities will be constructed (Fig. 7.3.6). Where facilities outside the dam itself are to be constructed, e.g. a powerhouse, an inlet structure, a chute, or a crane runway, surveys of profiles along their axes and in cross-sections are made. This part of the survey should also be carried out in sufficient detail to allow for changes that may be made in the position of facilities.

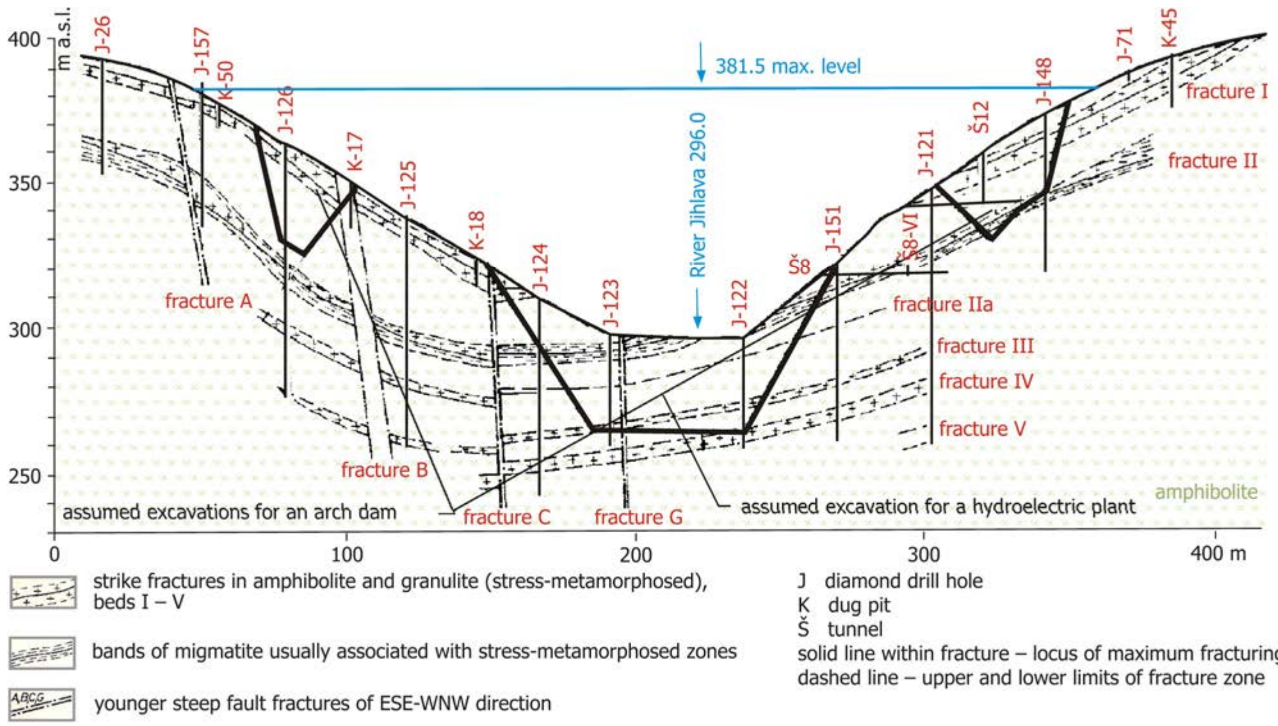


Fig. 7.3.6 Survey work in the Dalešice dam profile for the option of an arch dam

In valleys where access is difficult, a system of access roads must be constructed before the start of a survey. Sometimes this involves excavating long, deep cuttings down to the bedrock. These provide an opportunity to observe and describe the rocks and structures underlying the site and this should be one of the first tasks undertaken by the engineering geologist. For example, in the survey for the Caracusey PSHEP (1984), several kilometres of access roads were built and, in some places, cuts several metres deep were excavated. All these artificial exposures were progressively surveyed geodetically, geologically, and geophysically and also photographed systematically. These road cuttings are also suitable places from which to collect samples for petrographic description and various geotechnical tests. In some cases, unfortunately, the information obtainable from such exposures is not used to full advantage.

The depth to which survey work is carried must comply with the specifications for the design of the dam and ancillary facilities and provide the information necessary to carry out the construction of them. At the preliminary stage of the survey, the basic engineering-geological conditions governing the construction of the dam should be established. This means that subsurface exploration must be carried to a depth at which it is certain that all the geological factors that affect the safe construction and operation of the dam have been taken into account.

Among these factors are: deformation of the rocks due to changes in the field of stress, the location of permeable zones, mechanical or chemical suffosion, upward pressures of groundwater, presence of chemically aggressive water, etc.

In the survey of a dam site, it is also necessary to give attention to the investigation of the distribution of natural stresses in the rock mass and to assess the changes that will be caused as a result of construction and commissioning of the dam. From experience, the zone of concentration of stress occurs at the foot of the slope at a depth of 10–40 metres (Figs. 7.5.2 and 8.5.2). This zone is of particular interest to engineering geologists because it is usually characterized by reduced permeability.

This review of the geological circumstances governing the scale of subsurface exploration work required when carrying out an engineering-geological survey of a dam site shows that it is a complicated task. The approach adopted depends on many factors such as the complexity of the engineering-geological conditions at the site, the type and purpose of the dam, the height of the dam, etc. For these reasons it is not possible to advocate a rigid scheme for planning such work. In some cases this could have unfortunate consequences. An attempt to standardize the procedure for survey work was made in Cuba (1981). This led to excessive and thoughtless overscaling of the survey work. In a later edition of the proposed standard, a much simplified and non-prescriptive scheme was adopted. This is given in Tab. 7.3.1.

Similar guidelines were compiled in the former Soviet Union. In Table 7.3.2, recommended average distances between exploratory holes are given in relationship to the category of engineering-geological complexity of a surveyed area (Tab. 3.3.1) and for different design stages according to the height of the dam.

Table 7.3.3 gives recommended depths of exploratory workings on the basis of the same criteria. It shows that, in the stage of the preliminary survey for master and implementation projects, the guideline allows for shorter exploratory holes. The reason of this is that the engineering-geological conditions at a dam site should be established at the stage of the preliminary survey. At the stage of the detailed survey, exploratory workings should serve mainly for various special tests and

Table 7.3.1: Borehole spacing and depth in relationship to designed dam height

Category of engineering-geological complexity of area (see Tab. 3.3.1)	**Borehole spacing**			**Borehole depth**	
	H < 25	**25 < H < 50**	**H > 50**	**H < 50**	**H > 50**
A	80–200	70–150	50–100	0.6–1.4 H	0.6–1.0 H
B	60–150	50–100	40–80	0.7–1.6 H	0.7–1.2 H
C	40–100	30–80	25–60	0.8–1.8 H	0.8–1.4 H
H = maximum dam height					

Table 7.3.2: Recommended average spacing of exploratory holes in surveys for dams

Category of engineering-geological complexity of area	**Dam height [m]**								
	Project design			**Master project**			**Implementation project**		
	10–15	**16–50**	**> 50**	**10–15**	**16–50**	**> 50**	**10–15**	**16–50**	**> 50**
A	400	300	250	250	200	150	150	100	80
B	300	250	200	200	150	100	100	80	75
C	250	200	150	150	100	75	75	60	40

measurements in the area at the bottom of the footing within reach of effects caused by the construction itself. Even at the stage of the detailed survey, allowances are made for deeper boreholes to answer certain questions about the geological structure.

Table 7.3.3: Recommended average depths of exploratory workings in EG surveys for dam

Category of engineering-geological complexity of area	**Dam height [m]**											
	10	**15**	**20**	**25**	**30**	**40**	**50**	**60**	**70**	**80**	**90**	**100**
	Project design											
A	15	20	25	30	30	40	40	45	50	55	60	65
B	20	25	30	35	40	50	55	60	70	70	80	85
C	25	30	35	40	50	60	70	80	90	90	100	105
	Master and implementation project											
A	10	15	20	15	20	22	24	26	28	30	32	35
B	15	20	25	20	22	24	26	28	30	32	34	40
C	20	25	30	25	27	30	32	35	37	40	42	45

A number of observations can be made about the procedures to be used for the survey of dam sites and the way in which they should be carried out. Standards, strict procedures and formal guidelines cannot always be used in surveys of dam sites. This is because dams are designed to meet specific requirements imposed by geographical location, climate and topography, as well as the underlying geology. For example, in Argentina and India, recommendations for the depth of exploratory workings and the spacing between them have been made. In 1973, the Indian standard governing surveys for dams was issued and revised in 2006. This latest version of the standard recommends that an engineering-geological survey is divided into four stages (Tab. 3.4.1); recommendations for the type and extent of survey work to be carried out at each stage are also given. The basic survey methods recommended are geophysical work, direct survey by drilling, excavation of pits, trenches and tunnels, geotechnical work ***in situ***, and laboratory tests. The standard gives a detailed description of individual methods and the procedures to be used for the various types of work. A standard for engineering-geological surveys for dams has also been issued in the USA and consists of several subsidiary standards for applied geology, hydrogeology, geological mapping, various definitions, ecological problems, geological and seismological hazards, geotechnical work, and other topics.

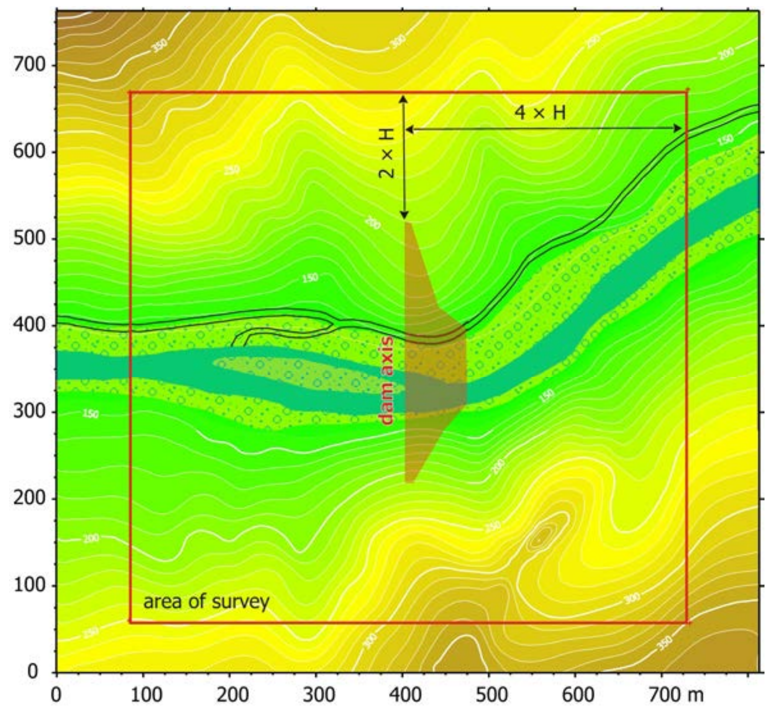


Fig. 7.3.7 Delineation of the area for a preliminary survey

The Japanese standard (1978) gives a practical definition of the area of a dam site, within which it is necessary to establish the engineering-geological conditions governing the construction of a dam for a future reservoir by carrying out subsurface exploration work. At the stage of the preliminary survey, the area is defined from the point of intersection of the dam axis and the ground at an elevation of the maximum reservoir level as follows: two dam heights (2H) outwards on both slopes, then from these lateral points a distance of quadruple the dam height (4H) upstream and downstream. This general rule applies providing that geological conditions are not so complicated that a special survey is required (Fig. 7.3.7). If geological conditions are complicated or the dam construction is unconventional, it is necessary to define the area in relation to the particular requirements of the client, the designer and the whole survey team.

At the stage of the detailed engineering-geological survey for a dam body and reservoir, the current Japanese standard delineates the studied area as follows:

- Outlines of the dam are plotted on a relevant topographic base;
- From different points of the dam outline, circles having the radius of the maximum dam height (H) are plotted and multiplied by the relevant coefficient as given in Table 7.3.4; and
- The curve forming the envelope of all the circles then delineates the area in which a detailed engineering-geological survey should be carried out (Fig. 7.3.8).

Table 7.3.4: Coefficients

Type of dam	Coefficient from dam axis		
	Upstream	**Downstream**	**Both banks**
A	1.0	1.5	1.5
B	1.0	1.0	1.0
C	0.5	0.5	1.0
Explanation:	*A: arch dams*		
	B: other concrete dams		
	C: embankment dams		

The Japanese standard defines the extent of the area surrounding a dam profile that should be covered at the preliminary and detailed stages of an engineering-geological survey. In the detailed survey, the maximum recommended depth of boreholes is equal to the dam height H. However, in contrast to the standards given above, there is no recommendation for the number of boreholes and the spacing between them. Instead, graphs showing the number of exploratory workings and their total length in relationship to the height and type of dam are given (Figs. 7.3.9 and 7.3.10). These graphs were modified and combined by Geotest to make them more useful for practical application. At the same time, an attempt was made to express individual relationships mathematically. It is natural that the degree of correlation is different for the separate relationships observed. This is due to the fact that the number of direct exploratory workings and their depth are influenced by a large number of factors, some of which have opposite effects. By using this procedure, however, a certain idea of possible patterns can be obtained.

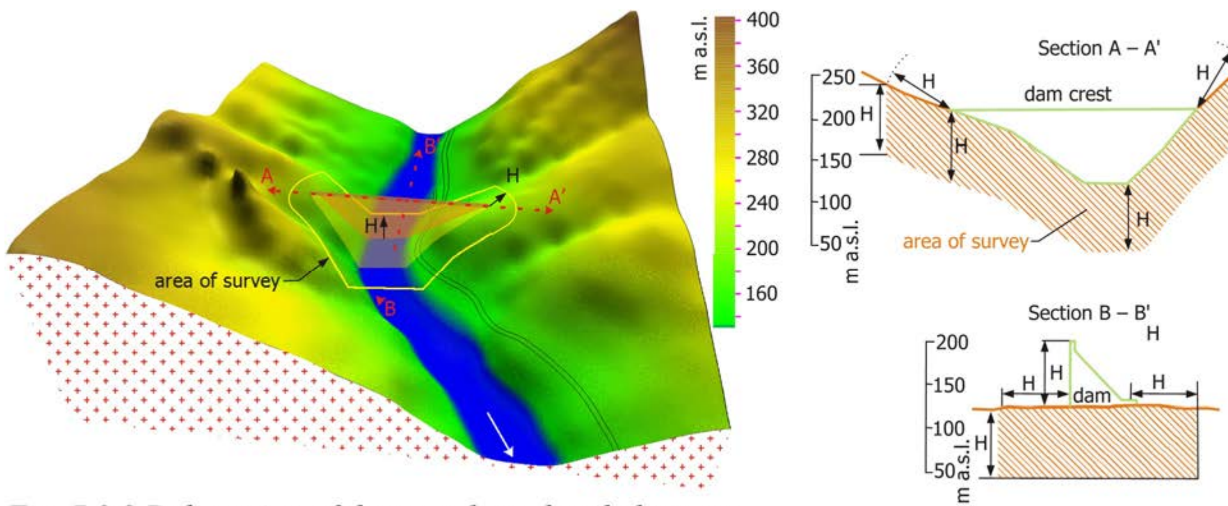


Fig. 7.3.8 Delineation of the area for a detailed survey

For the purpose of establishing relationships, dams have been divided into four groups. Even though this division is simplified, the graphs and equations obtained provide a large amount of interesting information. The examples show that it is difficult to standardize the number of exploratory workings. Even though there is a certain relationship between variables (the greater the dam height – the larger the number of exploratory workings), the graphs show a number of examples of low dams where it was necessary to use an enormous range of exploratory workings, whereas in other cases the number of workings was minimal even when it was a high dam. The graphs show that pits are commonly dug and used to a large extent in surveys of dam profiles, but in the Czech Republic this traditional and very valuable type of exploration work is in decline.

As far as the maximum depth of boreholes is concerned, the indications are more consistent. In order to assess the usefulness of the standards and recommendations cited in the literature, the relationship between the designed dam height and the maximum or average depth of boreholes has been determined based on surveys of twenty reservoirs carried out in the Czech Republic (Fig. 7.3.11).

Even though all these surveys, apart from the detailed survey carried out at Dalešice, were

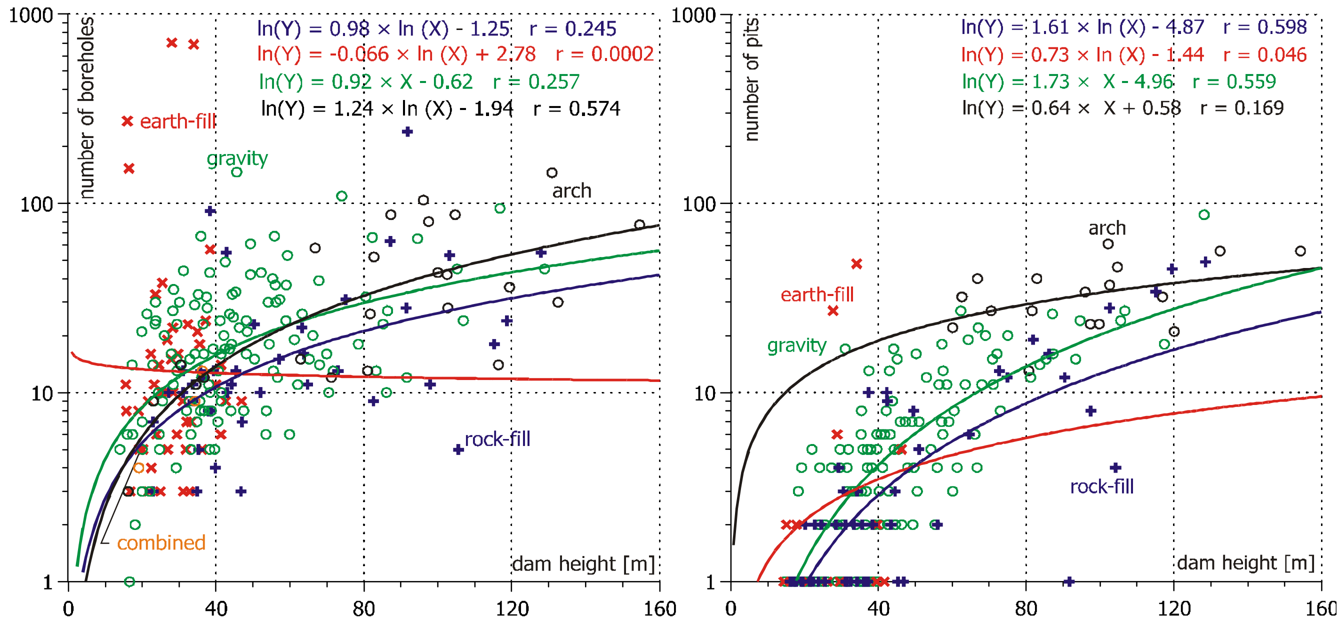


Fig. 7.3.9 Relationship between dam height and the number of boreholes and pits

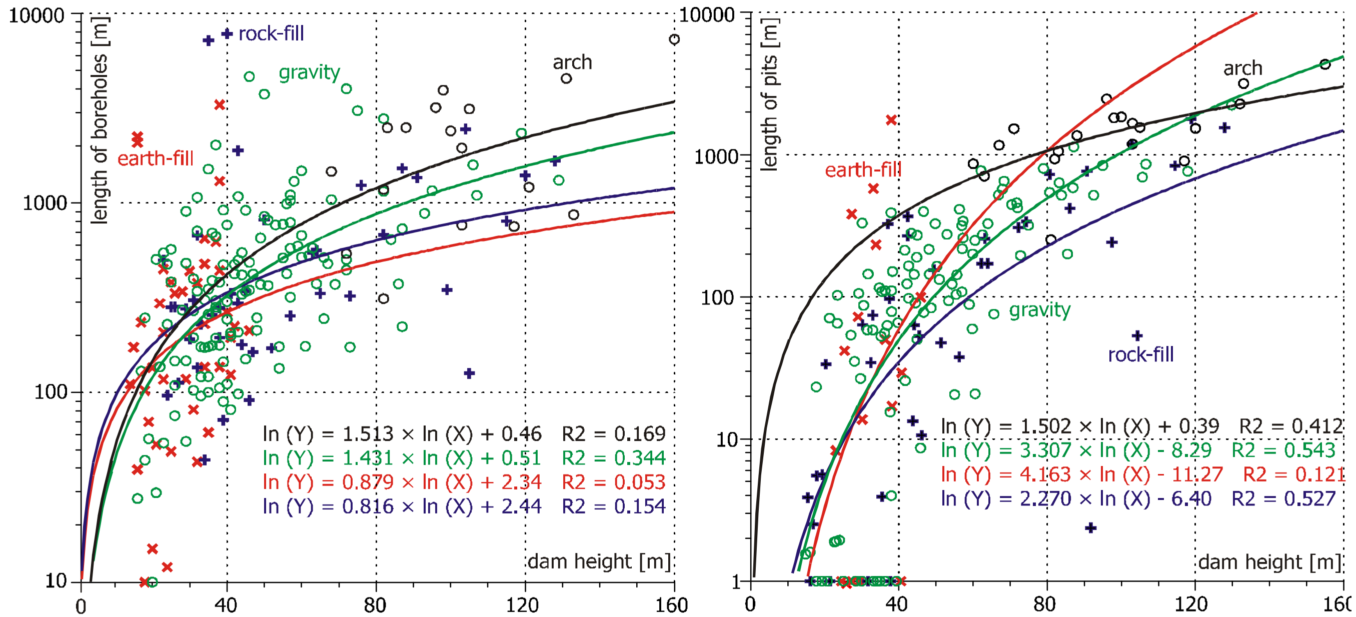


Fig. 7.3.10 Relationship between dam height and the total length of boreholes and pits

made at a preliminary stage for the project design, the maximum depths given can be considered as representative. At this stage of a survey, when it is necessary to establish the geological structure of a site and determine the permeability of the rocks, the depths of boreholes should be greater than at later detailed stages of the design, when attention is focused on establishing the conditions of the foundations beneath the planned dam and ancillary facilities and on carrying out special tests in boreholes. In Table 7.3.5 the relationships between the heights of dams and the recommended maximum depths of exploratory holes is shown.

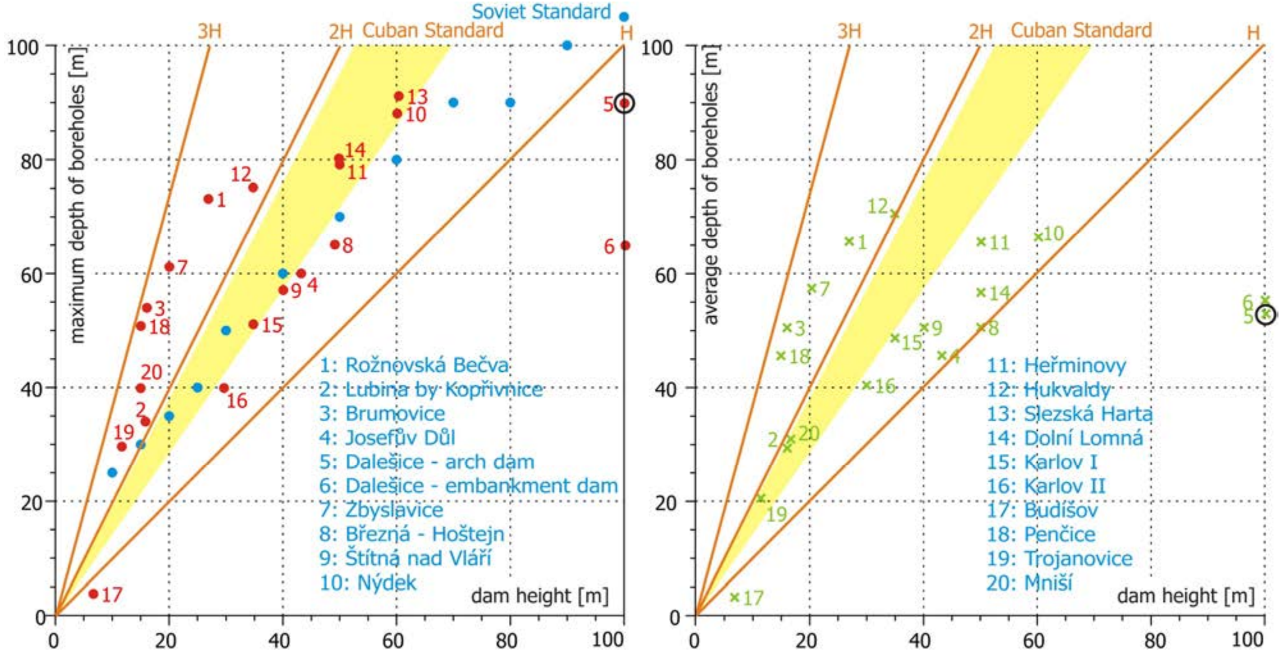


Fig. 7.3.11 Relationship between dam height + maximum and average depths of boreholes

A comparison of the individual standards with the results of the evaluation of twenty examples of Czech dams and reservoirs shows good agreement, especially for dams higher than 30 metres. For dams lower than 30 metres, the following conclusions can be drawn. Either the rocks in the Czech Republic and Slovakia are intensely fractured and permeable to such great depths that it is necessary to drill to depths of about 40–80 metres even in the case of low dams, or too much emphasis was placed on ensuring that the rocks in the foundations were not permeable. If the recommendation of J. Verfel (1974) to adopt the Lugeon criterion for depths down to 10 to 15 metres was accepted, Czech practice would match the recommendations given in foreign standards.

Table 7.3.5: Maximum recommended depths of boreholes used in EG surveys for dams

Dam height [m]	CSSR	Cuba Tab. 7.3.1	USSR Tab. 7.3.3	Japan Argentina	Remark
0 – 20	2H–3H	1.4–4.8H	1.8–2.7H	15 m	A maximum of 3H is recommended in the literature
20 – 30	1.4–2.8H	1.4–1.8H	1.6–1.8H	0.5–1.0H	
				1.0–1.5H	
30 – 60	1.3–1.8H	1.4–1.8H	1.3–1.7H	0.5–1.0H	
				1.0–1.5H	
60	0.6–1.0H	1.0–1.4H	1.0–1.4H	0.5–1.0H	
	0.8–1.4H			1.0–1.5H	

Note: Upper line: embankment dam, lower line: concrete arch dam
The Soviet standard gave recommended average depths for holes

The facts reviewed above show that the recommended depths can be used as a guide in surveys for different heights of dams. In doing this, it is necessary to proceed so that the lowest values given will be used in areas that fall in category A of engineering-geological complexity, the intermediate values in category B, and the highest values in category C (according to Tab. 3.3.1), bearing in mind the specific features of each dam site.

According to the standard used in the former USSR (Tab. 7.3.2), the recommended spacing between boreholes used in an engineering-geological survey for a project design for all dam heights and various categories of engineering-geological complexity was from 150 to 400 metres. According to the Cuban standard, which does not stipulate a design stage, the recommended spacing is from 25 to 200 metres. The evaluation of several examples of Czech dams (Figs. 7.3.11 and 7.3.12) shows the recommended spacing between boreholes to be from 75 to 240 metres, and occasionally more (dam No. 10) or less (Nos. 11, 14, 15, and 17). If this level of exploration of a dam site is compared with the former Soviet standard, then it corresponds more with the state of exploration reached at the master stage of a construction project (Tab. 7.3.2 gives the spacing between boreholes ranging from 75 to 250 metres).

From the examples given it can be appreciated that, in former Czechoslovakia, more attention was given to the engineering-geological survey at the project design stage.

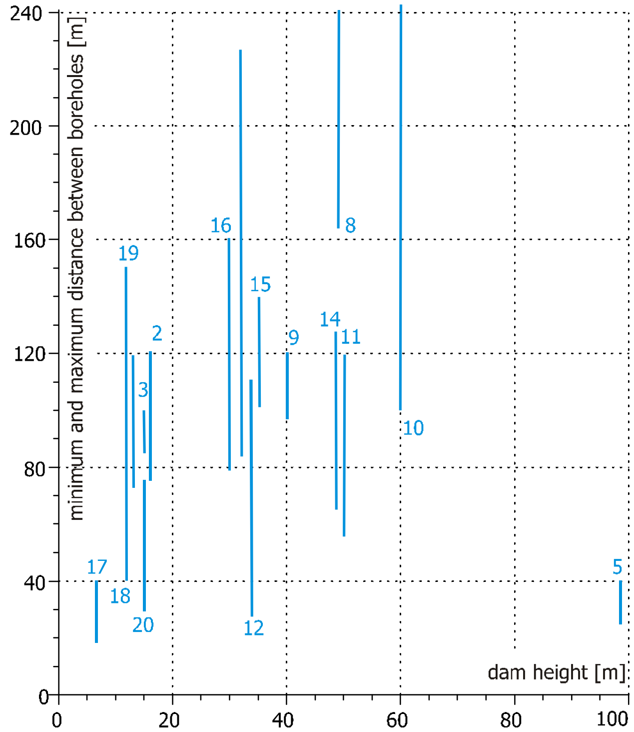


Fig. 7.3.12 Maximum distances between boreholes (for identity of profile – see Fig. 7.3.11)

Bearing in mind that this chapter deals with the scope of subsurface exploration work carried out during an engineering-geological survey, it is also necessary to give details of the depths of boreholes required when making water pressure and grouting tests. J. Verfel (in Verfel, Tkaný, 1974) states that the depth of exploratory boreholes must be greater than the designated depth of a grout curtain. The depth of a grout curtain was formerly designed to be equal to the height of the reservoir water level, or at least 2/3 of this height. The American Bureau of Reclamation uses the following formula for determining the recommended depth of a grout curtain:

$$\mathbf{s = 1/3\ H + C},$$

where: **s** is the depth of the grout curtain;
H is the height of the reservoir; and
C is a constant (an increment of depth that varies according to the complexity of the geology at the site).

Different authors have determined the constant in different ways, e.g., Simons gives C within the range of 8 to 23 metres. The constant was determined according to the complexity of geological conditions at a dam site and in relationship to the dam height.

There have been numerous cases in which a grout curtain was not designed to be sufficiently deep (e.g., Hoover Dam, Koryčany, etc.). These have proved that there is no simple formula that can provide a satisfactory answer. The appropriate depth for a grout curtain can only be determined by systematic drilling and water pressure tests. The depth of exploratory boreholes must be such that the measured rates of loss of water do not exceed the Lugeon criterion (in dams higher than 30 metres, 1 l/min/m at a pressure of 1.0 MPa; in dams lower than 30 metres, 3 l/min/m at a pressure of 1.0 MPa).

In the Czech Republic, the Jähde criterion is often used. This permits losses of water amounting to 0.1 to 0.5 l/min/m at a pressure of 0.3 MPa and it is usually required that this criterion is met by three successive exploratory storeys of at least three metres in length. In all cases, it is necessary to conduct water pressure tests down to a depth at which the rocks can be shown to be safely impermeable according to the criterion of Lugeon or Jähde. Based on a study of water flow through the rock in the subsoil of a dam, and bearing in mind the decrease in the gradient in the curtain with increasing depth (Fig. 7.3.13), J. Verfel (in Verfel, Tkaný, 1974) recommends that maximum attention should be paid to the footing of dams and to the underlying rock down to a depth of 10 to 15 metres.

At depths greater than 15 metres, where the value of the gradient decreases by a factor of four, it is possible to accept larger losses of water, up to 2 l/min/m at a pressure of 0.3 MPa; below a depth of 30 metres, where the value of the gradient is up to ten times less, it is possible to accept losses of water of up to 4 l/min/m at a pressure of 0.3 MPa. Below a depth of 50 metres, it is necessary to design a grout curtain so that the water consumption in inspection boreholes is not larger than 6 l/min/m at a pressure of 0.3 MPa. J. Verfel (in Verfel, Tkaný, 1974) states that these values apply particularly to dams lower than 30 metres. In addition to water pressure tests, there are other important factors that must be taken into account. These include the type of rock (bearing in mind the resistance of the rock to disintegration by water pressure), the types of filling in joints (bearing in mind the possibility that fillings may be washed out resulting in the development of underground suffosion), the dip of bedding planes, the orientation of the main joint systems and faults (these structural features influence the pattern of water pressure contours in the rock), the depth of weathering, weathering along joints, the height and type of construction of the dam, the function of the dam, etc. It is evident that if certain rates of seepage that do not threaten the safe operation of the dams are permitted, it is possible to install a shallower curtain or not to install a curtain at all. Conversely, if seepage is small, but upward pressures are large enough to threaten the stability of the dam, it is necessary to extend the curtain.

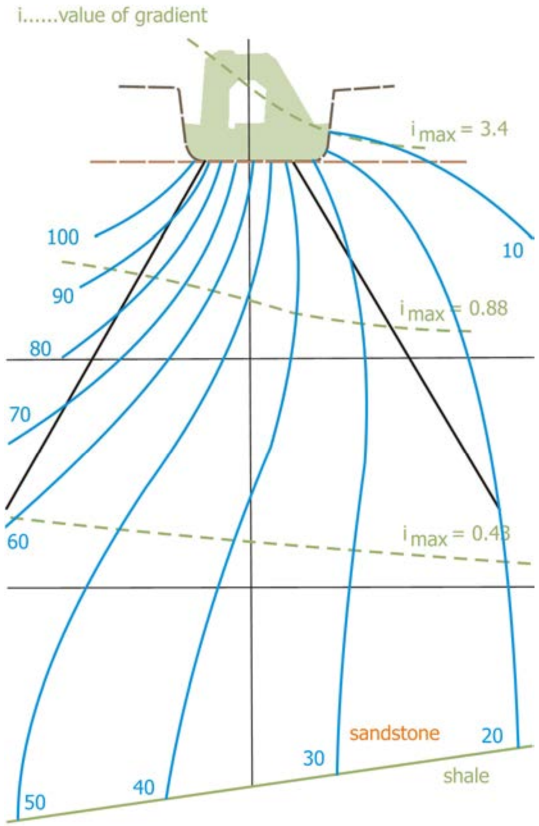


Fig. 7.3.13 Pattern of lines of same pressures (Verfel, 1974)

To determine the spacing between boreholes in the curtain and to select the appropriate grout mixtures and calculate the amount of material required, a grout test must be carried out.

The facts presented above show that it is not possible to define the depth of a grout curtain and the depth of exploratory boreholes simply on the basis of the permitted permeability of rocks in the subsoil of a dam. It is also necessary to take into consideration the velocity of water flow in the subsoil and to determine the critical pressure above which joint fillings are washed out or the rocks are disintegrated. In the literature it is stated that in semi-solid rocks in a weathered state, rocks can be disintegrated if the hydraulic gradient is greater than 10, and soils will be broken down at even lower values.

7.4 Comprehensive Documentation of Exploratory Workings

The standards set for the quality of information put into the database for a dam project are demanding. Detailed and complete information about the geological composition and structure of the site must be obtained as well as comprehensive descriptions and measurements of the geotechnical properties of the rocks. Much of this information will be acquired by observation and sampling in direct exploratory workings. These may be tunnels, pits, hillside cuttings, boreholes, temporary exposures or road cuts, etc. The quality of the geological and petrographic description of rocks exposed in exploratory workings depends on the practical experience of the engineering geologist and the facilities he has at his disposal. In many cases, description and sampling of the complete length of an exposure may not be possible, either because of the incomplete recovery of core from a borehole or because the circumstances require fast and continuous casing of tunnels or pits, or because of other reasons.

When studying the geotechnical properties of the rock mass by means of field and laboratory tests, data is obtained by making observations and measurements at surveyed points in underground exposures or by collecting samples. This is point information. To enable a more accurate application of this point data to the whole rock mass, new techniques making use of indirect, non-destructive methods of measurement must be used. Rapid development of indirect methods of documentation has taken place since the 1970s. These include geophysical, geomechanical and photographic techniques. Certain conclusions and recommendations for the comprehensive documentation of exploratory workings and boreholes have arisen from research carried out by Geotest, notably in connection with the survey of dam sites in the former Czechoslovakia and abroad. These can be applied when carrying out similarly demanding surveys for engineering constructions.

A preliminary stage in the comprehensive documentation of direct exploratory workings is to make parametric measurements in the vicinity of the planned excavations. For this purpose, geophysical sounding methods are particularly useful, most often vertical electrical sounding and shallow seismic refraction. Parametric measurements are made in vertical exploratory workings. Generally, they are not made in horizontal or oblique workings. The illustration in Figure 7.4.1 is from the Šance – Řečice site, where survey work was carried out on the reservoir site of a flat-lying landslide.

In order to obtain a better idea of the distribution of velocities, the individual SSR travel-time curves were interpreted separately and average values of depths and velocities were subsequently determined for all four travel-time curves. Similarly, the individual directions of VES stretching were interpreted and the average depths and the average resistivities were also determined. The results of the interpretation of the seismic work enabled the boundary between slope silty clay and loamy-stony debris to be successfully defined. In this case, it can be observed that the seismic boundary is usually a little deeper than the geological boundary. This is due to the fact that the uppermost part of the colluvial debris is rather similar to slope silty clay in its behaviour and properties. The results of VES satisfactorily define the boundary between slope silty clay and colluvial debris and, in addition, one internal boundary in the landslide. The base of the shear plane is clearly defined by the interpretation of the geoelectrical measurements.

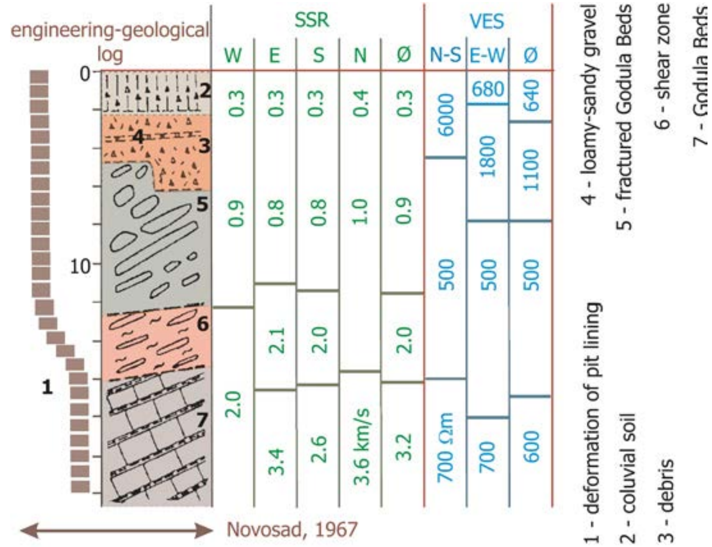


Fig. 7.4.1 Parametric measurements in a dug pit

Based on the results of parametric measurement given here, it is clear that geological boundaries cannot always be defined at all points of measurement. It is necessary to bear this in mind when planning geophysical work. The individual points at which measurements are made should be positioned as closely as practical, and if circumstances permit, measurements should be made in a cruciform configuration, and the success with which individual boundaries can be defined will increase.

7.4.1 Documentation of Tunnels and Pits

Tunnels and pits enable the observation, description and measurement of the rock mass *in situ*. The comprehensive documentation of such excavated workings can be divided into geological, geophysical, geomechanical and photographic.

Geological profiling is the standard method used for engineering-geological documentation. They are compiled in graphic form as sections and exploded block diagrams in which the boundaries between geological formations are depicted on the basis of mineralogical and lithological criteria; the type and orientation of geological structures such as faults and folds are also shown, together with the degree of fracturing and weathering of rocks, and the planes of mechanical discontinuity. The graphic depiction of exploratory workings is complemented by a key to the types of rock that have been identified, together with measurements of planar and linear features of the rock structure (strikes and dips of bedding, foliation and jointing, together with the plunge of mineral lineations and fold axes) given by the direction of dip and the amount of dip. The measurements are plotted on stereographic diagrams, processed statistically and supplemented by detailed information about the types of discontinuities in the rock mass. Based on these measurements and descriptions a classification of discontinuities is made using the following three categories:

- Geometrical: this allows a large number of systems of discontinuities to be classified into characteristic groups. For example, the geometrical classification of joints can be made according to their angle of dip as follows: vertical, with angles of 90–70 °, steep (70–50 °), oblique (50–6 °), and horizontal (0–6 °). A more suitable division of systems is made in terms of their spatial orientation in relation to bedding planes and foliation or to fold axes.
- Topographic: A topographic type includes all those discontinuities having the same shape. The topographic type can include several systems with different orientations, just as one system can incorporate different topographic types. And
- Genetic: A genetic type includes all those discontinuities formed as a result of the same geological process (e.g., lithogenetic, tectonic and exogene joints).

The classification of joint systems is especially demanding and an engineering geologist must exercise great skill to do it properly. Sometimes this is not possible without collaboration from other geologists specializing in tectonics, metamorphism or other branches of geology. At the site of the Dalešice PSHEP on the River Jihlava, the rock mass underwent several important metamorphic and tectonic events during which the earlier-formed joint systems were reactivated, transposed and otherwise deformed. Therefore, in this case particularly, it was necessary to study the genesis of joints in detail and to assess their continuity and frequency. In all, eleven systems of fractures or joints were identified at this site. These were genetically related to the planes of metamorphic foliation, fold structures, strike-slip faults and to younger fault tectonics. In addition, a system of young sub-horizontal joints was identified. These were inclined 5 to 15 ° downslope, and continuous over several tens of metres. These joints intersect older joint systems and cause displacements of the order of centimetres. They obviously originated due to the relief of residual stresses in the rock mass as the rocks were exposed by erosion.

Documentation of tunnels and pits is either made in one stage or in two stages (preliminary and detailed). Observation and description is carried out in one stage in workings which must be continuously lined because of instability or other reasons so that there is no guarantee that there will be access to the exposures in future. The preliminary record is made as the tunnel is driven. In order to be effective, the engineering geologist responsible for the preliminary description should already be familiar with the main features of the geology and structure at the site. This will enable the best line to be chosen for the tunnel itself and the best positions selected for blind cross-cuts and sampling. Geological problems can be anticipated and the best sites for field tests can be selected. At the stage of preliminary documentation, the faces of the advancing tunnel and the floors and walls of pits are measured and described at each stage of advance.

Detailed mapping and description of a tunnel is not made until it is completed and the walls and back have been washed and cleaned, lighting has been installed and the tunnel has been properly surveyed. Graphic and written records of the geology will be made using the geodetic survey for control. Mapping of the walls and back is usually carried out at a scale of 1:50, and at some especially important sections, e.g., where field tests will be made, plans may be made at a more detailed scale (e.g., 1:20). It is general practice to map both walls of a tunnel and also, sometimes, the back. In certain cases geological features in the floor of the tunnel must also be recorded, particularly the position of fracture zones and important geological contacts. Because the surfaces of a tunnel tend to be rather irregular, it is necessary to project the observations into an ideal plane passing approximately through the middle of the irregularities.

In the late 1960s, Geotest adopted the practice of combining conventional geological mapping and description with geophysical measurements on the floor of exploratory tunnels. The reason was the need to obtain basic data for the interpretation of subsurface geophysical measurements and the access to exposures that would enable a more complete description of the rock mass and to choose and prepare the best sites to make geotechnical tests. It is natural that geophysical documentation has improved over the years. This is due to the fact that experience has been acquired progressively and that the procedures are now regarded by engineering geologists, geotechnicians and designers as standard.

Figure 7.4.2 is an illustration of measurements made in tunnel Št14 at Dalešice. In those days, not only geoelectrical and seismic methods, but also measurements of magnetic susceptibility and radiation exposure rate were being used for the first time. In addition to these geophysical measurements, geotechnical measurements were also made on the walls of the tunnel. The style of documentation described was used not only in tunnel Št14, but also in the other tunnels driven at the site.

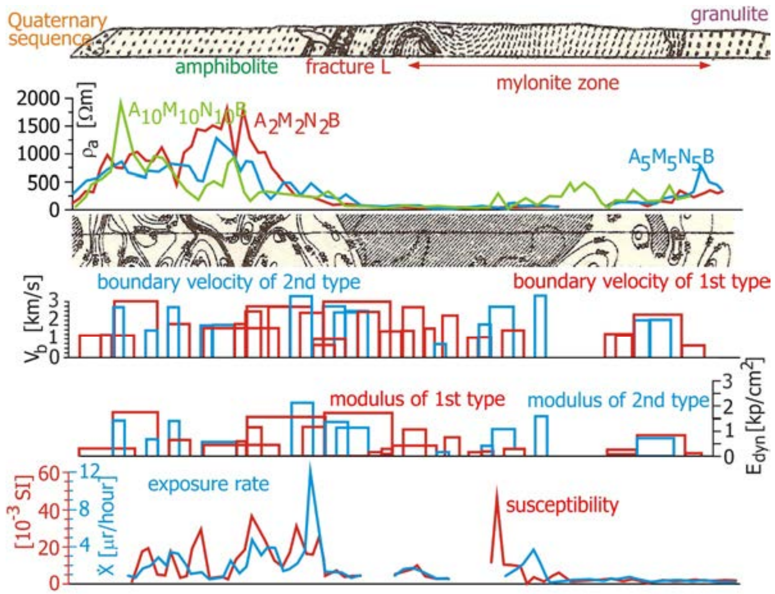


Fig. 7.4.2 Geophysical logging of a tunnel, showing the methods used in the late 1960s

The procedures used in fieldwork and for processing the observations and measurements that were made were dependent on the knowledge and instrumentation available at that time. As has already been pointed out, this was particularly true in the case of seismic work, because, at that time, seismic profiling was gradually being replaced by improved methods of continuous seismic sounding. This enables the velocities of seismic waves to be determined more accurately. This technique can compensate for the effect of the dip of the seismic boundary in the interpretation of results.

At that time, it was usual to express the results of geoelectrical measurements in the form of contours of apparent resistivity. This procedure was also maintained when measurements were made along the floor of a tunnel. This method was gradually abandoned and only recording in the form of graphs has remained. The reason is that measurements made in a tunnel are a form of geophysical logging as opposed to conventional measurements made on surface. This applies especially when wider spacing of electrodes is used. Therefore, the conventional 2 π geometry (half-space) was replaced by the 4 π geometry (full space). A problem occurs in cases where the diameter of the tunnel is not negligible in relation to the distance between measurements. In these cases, it is necessary to calculate the constant of spacing when changing from the 2 π to 4 π mode using special coefficients for correction. These are given in the literature, for example, by Khmelevskoy (1975).

In the case of geoelectrical measurements, care must be taken when evaluating the minima of apparent resistivity. In tunnels, it often happens that the floor does not coincide with the surface of the rock mass, but that the rock is covered by a variety of fine material – a mixture of silty clay, the clayey filling from joints and fault gouge, and water. It is natural that this material will cause a definite reduction in the measured values of apparent resistivity, and in extreme cases, the electric current is conducted not only through the rock mass, but also through rails on the tunnel floor. When the results of geoelectrical measurements are interpreted by an inexperienced geophysicist or engineering geologist, it may happen that the fracturing of the rock mass is described as being more intense than it is in reality. Later on, for this reason, the use of conventional surface penetration electrodes was abandoned and contact electrodes were used to make closely spaced measurements along tunnel walls.

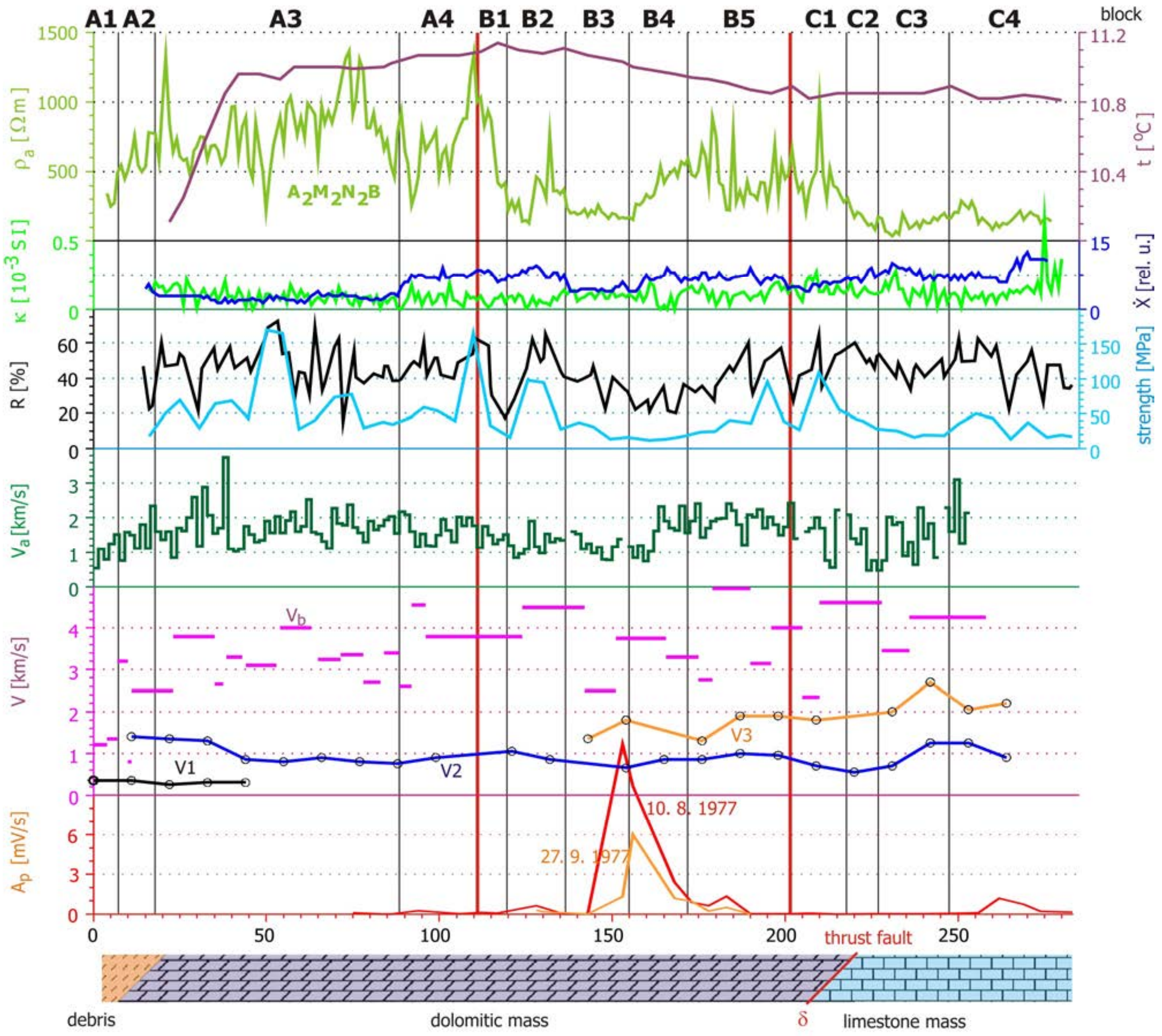


Fig. 7.4.3 Comprehensive geophysical logging of a tunnel

Figure 7.4.3 shows the comprehensive documentation of a tunnel as carried out in the 1970s and 1980s. The illustration is from the Tereza tunnel at the Malá Vieska site. In addition to the methods described above, geoacoustic measurements and a contact thermometer were used in the survey of the tunnel. In both cases the measurements were made in short holes on the floor of the tunnel.

Geological observation showed that the section of the tunnel up to 210 m consisted of dolomite and that the remainder of the tunnel was in limestone. Based on the comprehensive measurements made in the survey of the tunnel, it was possible to divide the rocks exposed in the tunnel into three main blocks: A (0 to 115 m), B (115 to 210 m), and C (210 m to the end of the tunnel), i.e. two blocks consist of dolomitic rock and one of limestone. After a more detailed analysis of all the measurements, it was possible to sub-divide the main blocks. The locations for geotechnical tests were chosen using the measured data, and other parameters were derived directly by calculation from geophysical

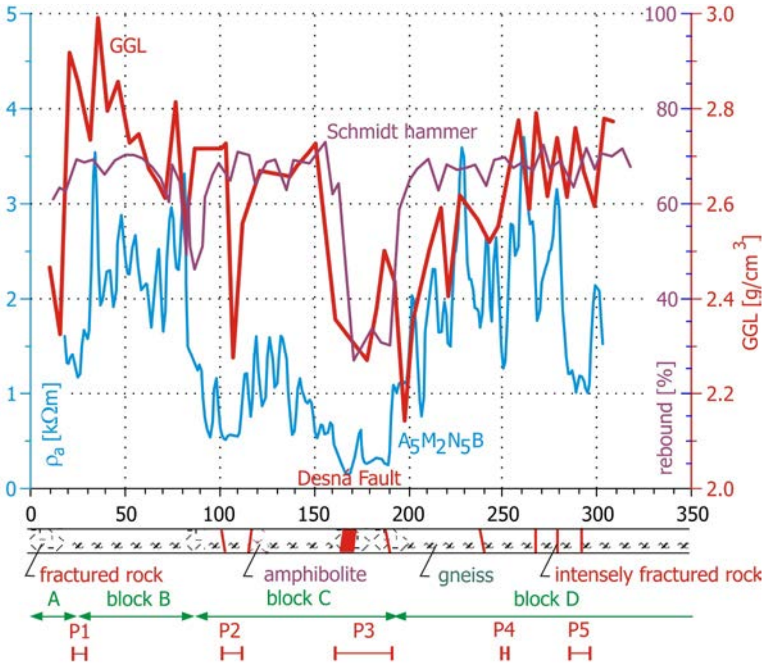


Fig. 7.4.4 Log of bulk density along a tunnel

measurements (Poisson's ratio, moduli of elasticity, etc.). The geoacoustic anomaly detected at a distance of 150 metres along the tunnel is interesting. At first, no direct geological explanation for this anomaly could be found, so the measurement was repeated with another apparatus after approximately seven weeks. The existence of the anomaly was fully confirmed and it was realised that it was caused by a zone above a thrust plane in which stress was concentrated. In this case dolomite was thrust over limestone.

If necessary, nearly all the geophysical methods used on surface can be used for surveys of tunnels. The following illustration is from the Dlouhé Stráně site, where measurements of bulk density were made in addition to routine geophysical measurements. Using the gamma-gamma method, measurements were made in holes two metres deep in the floor of a tunnel. The interval between measurements in the holes was 0.5 metre, and the space between holes was five metres. In Figure 7.4.4, apparent resistivity and the results of Schmidt hammer rebound tests are shown in addition to the measurements of bulk density. The measurements of bulk density gave rather surprising results, both in terms of the average value and the large difference between the maximum and minimum value. The most striking anomalies lie on the Desná Fault where the bulk density decreases to below 2.2 g/cm^3. This decrease is not at all surprising in such intensely fractured rocks as those around the Desná Fault. Frequent fluctuations of values from one to two tenths can be attributed both to errors in measurement, and to variable fracturing of the rock mass caused by blasting. When driving the tunnel, the charges used to break the rock were often more powerful than necessary.

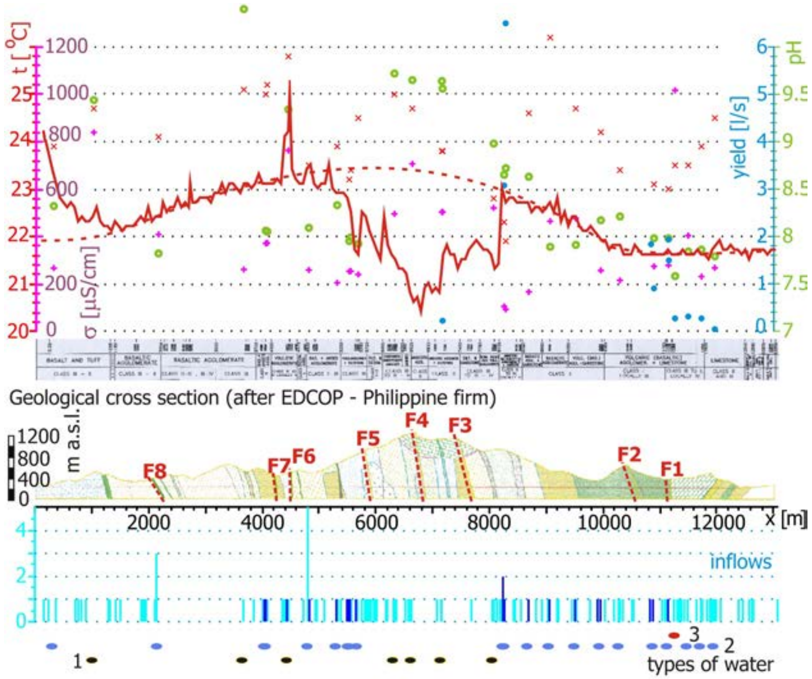


Fig. 7.4.5 Measurement of temperature and properties of water inflows along a tunnel

When surveying the Angat tunnel, which had been in operation for seven years, contactless measurements of the temperature, pH and conductivity of water flowing into the tunnel were made (Fig. 7.4.5). A portable pH-meter was used, so that all measurements could be carried out directly in the tunnel during its regular daily shutdown.

The lowest part of the figure illustrates the types of groundwater which were identified using cross graphs of the relations between individual parameters shown in the middle section of the figure. The locations where inflows of groundwater to the tunnel occur are plotted above the types of water. The dark blue colour denotes the places where the rates of inflow were estimated to be greater than about 1 l/s, the light blue colour all the other inflows. The largest of the 11 measured inflows was at 8,272 metres where about 6.5 l/s flowed into the tunnel.

All the measured parameters show that there are considerable differences in the properties of the groundwater. The most striking variations occur in pH. The values change from pH 7.58 at 11,258 m to pH 10.4 at 3,656 m. The temperature of the inflowing water ranges from 21.9 °C at 8,274 m to 26.2 °C at 9,056 m. The minimum conductivity of 92 µS/cm was detected at 8,274 metres along the tunnel, and the maximum value of 1,017 µS/cm was measured at 11,258 metres.

Comprehensive documentation of pits is not made so often, but this procedure is capable of providing information that it would be difficult to obtain in any other way. Figure 7.4.6 shows the measurements made in a pit at the Hrhov site. The pattern of the apparent resistivity curve matches the predictions made about the geological section before the pit was surveyed. The resistivity of the silty clay is logically lower than that of the limestone debris. The results obtained from seismic measurements were still more interesting. The absolute values of the velocities of the longitudinal waves were not as surprising as the non-uniform pattern of times. In certain places, the travel time decreases with depth, which is not what would be predicted. The only possible explanation lies in the distribution of cemented limestone debris that is not horizontally bedded. The values of velocities reached over 4 km/s in these beds which is a relatively high value. The decrease in travel time should not have occurred if the beds were uniformly horizontal so it was deduced that the horizontal units were connected by vertical zones. In this case, the seismic wave can propagate along paths of shortest time, and not along the geometrically shortest ones. This effect is very well known in seismic tomography. Above each cemented horizontal bed, a discontinuity that prevents the seismic wave from propagating in all directions must be present. This is the only explanation for the complicated pattern of travel-time curves. The geotechnical conclusions to be drawn from these observations are that the rock mass is much more anisotropic than would have been predicted on the basis of geological observations alone, and that there are many planes of discontinuity within it.

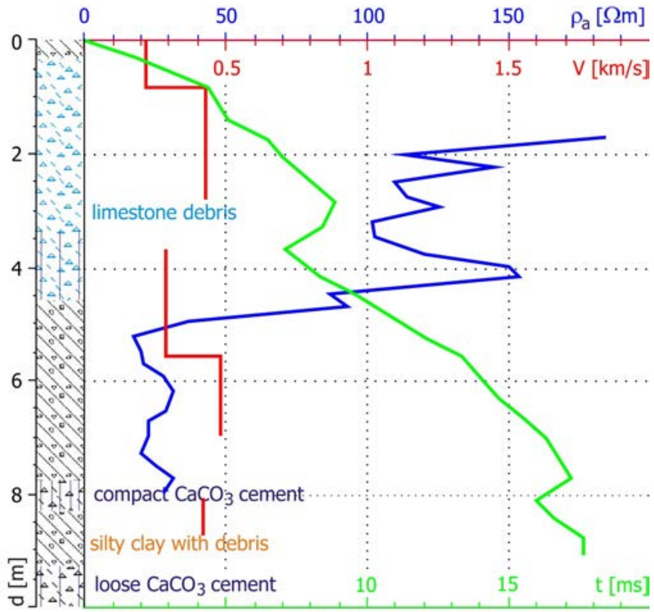


Fig. 7.4.6 Comprehensive logs of a pit

In conclusion, the various methods used in geophysical surveys of exploratory excavations, particularly of pits and tunnels, provide valuable information about the following:

- Lithological characteristics of the different types of rocks;
- The location of weak zones within the rock mass; and
- The geotechnical properties of constituent blocks of the rock mass.

The advantages of the individual geophysical methods are as follows:

- Geoelectrical methods contribute to the knowledge of all aspects of the geology of a site;
- Seismic measurement is used mainly for determining the mechanical properties of rocks and materials;
- Measurements of apparent magnetic susceptibility help to distinguish acid rocks from basic rocks and also to delineate tectonically stressed zones with higher contents of ferromagnetic minerals;
- Measurement of the radiation exposure rate enables rocks with higher contents of potassium (and of thorium and uranium) to be distinguished;
- Measurement of temperature enables interpretation of the pattern of flow of groundwater; and
- Logging methods are used infrequently in underground engineering-geological surveys, mostly in the search for large cavities.

A surveyed tunnel or pit can be divided into quasi-homogeneous physical units and the most credible physical and mechanical parameters for these units can be determined from the analysis of the pattern of their physical properties. Geophysical measurements also enable the most representative sites to be chosen for *in situ* tests of mechanical parameters and the selection of samples for laboratory analysis.

The width of the disturbed zone around a tunnel can be determined both by measurements made on the walls, back or floor of the working, and by measurements made in boreholes. For measurements along the floor or sides of a tunnel, seismic refraction is used. For precise surveys in boreholes, high-frequency seismics, ultrasonic probes or heads, and gamma gamma measurements are used. Both these techniques of measurement enable the pattern of distribution of stress around the excavated workings to be determined with relatively high precision.

To complete the description of a surveyed working, details of the drift faces made during the preliminary stage are compiled. The drift face forms a section perpendicular to the walls of a tunnel and by following the changes in geology exposed as the tunnel advances, fracture systems and lithological boundaries on both the sides of a tunnel can be monitored. It is necessary to define the lithological characteristics of the different types of rocks, and to record them on successive sections using the same ornament. The extent and type of weathering and hydrothermal alteration must also be recorded, as well as the frequency of fractures. The dip and the dip direction of all planes of mechanical discontinuity are measured systematically, especially those of bedding planes and joints. When describing planes of mechanical discontinuity, it is important to record their frequency, continuity, smoothness, separation and infillings, to describe the morphology and character of their surfaces, and to note the relationship between joints and bedding or foliation planes and linear structures in the rock. It is particularly important to measure the orientation and frequency of various joint systems and determine their other characteristics. The next step is to establish the patterns of discontinuities in the individual parts (blocks) of the rock mass that are lithologically or otherwise distinct. These observations are analysed in the field and later using procedures that enable the different sets of joints to be classified into genetic categories. To do this, histograms, rose diagrams and contoured stereographic projections of the poles to the various joint sets are supplemented by statistical analysis using computers.

The graphic and written description of tunnels is a very laborious and time-consuming task and cannot usually be completed at the actual time of the construction and survey work. At these stages, detailed documentation would impede the pace of construction and, at the same time as an advance is made, the tunnel is also being lined. For these reasons, Geotest usually makes a continuous photographic record of the exposures as they are created in the advancing tunnel. Subsequent analysis of these photographic images provides information about the strike and dip of beds and their thickness, the attitude of important geological contacts, folds and faults and about the orientation of joints and their separation and the lengths of structural features. All this information can be analysed statistically. This approach was used for the documentation of the geology of the diversion tunnels constructed at the Dalešice dam site.

The tunnels and pits excavated during the engineering-geological survey for the Dalešice dam enabled a detailed study of the rock mass *in situ*. One advantage of the tunnels excavated at Dalešice was that it was not necessary to line the walls so they remained accessible for repeated investigation. This was necessary because the opinion of the engineering geologist concerning the geological structure of the area was progressively changing. In addition, ready access to underground exposures meant that the characteristic parameters of the rock mass could easily be determined. Geological observations were recorded in the form of maps and sections of the walls of tunnels and pits, showing the shape of important geological contacts, the frequency of fractures and the intensity of weathering, and the position of important planes of mechanical discontinuity. The graphic documentation was supplemented by measurements of the direction of dip and the amount of dip of planes of mechanical discontinuity and lithological boundaries, the separation between joints and descriptions of their infill, roughness and continuity. The measurements were processed statistically so that the different sets of joints could be identified in terms of the direction and amount of dip, the frequency and character of infilling, and their geometrical and genetic characteristics.

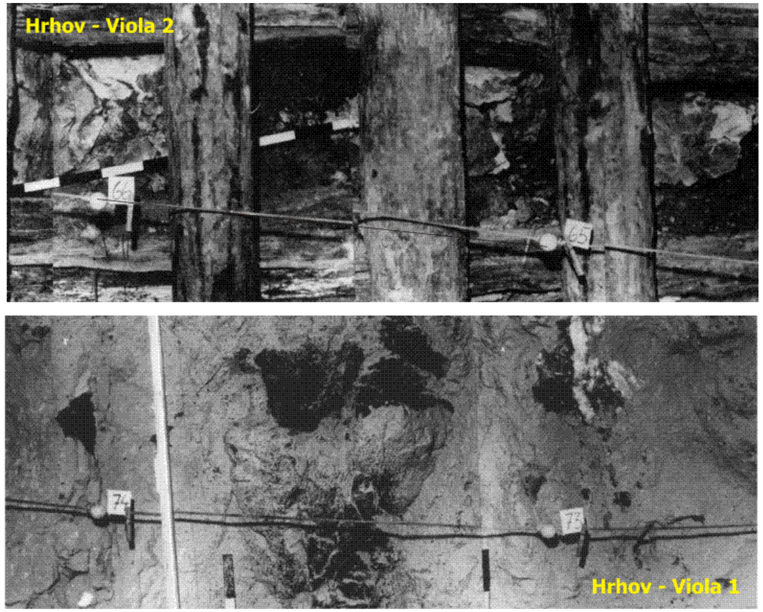


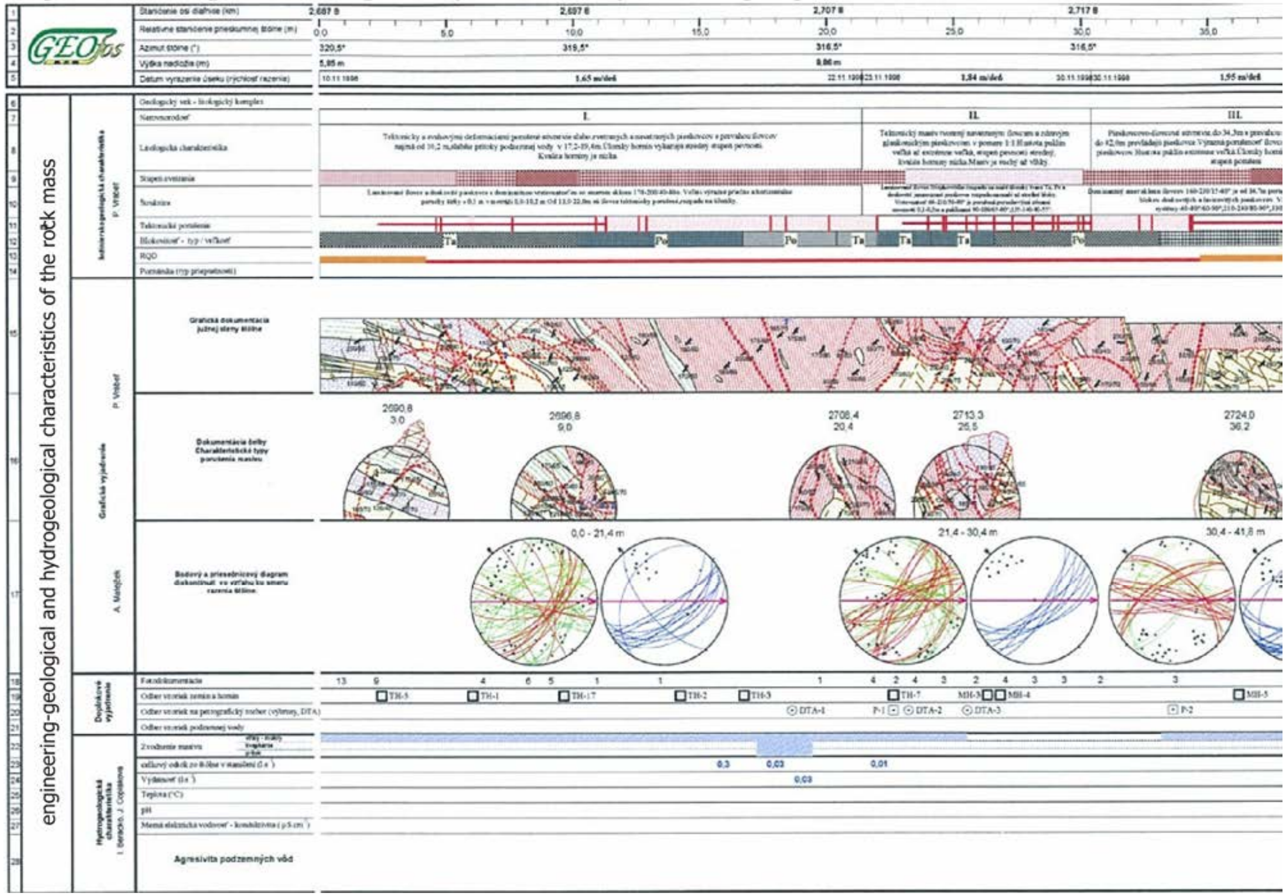
Fig. 7.4.7 Photographic documentation of a tunnel wall (Adamus, 1974)

Geomechanical profiling is based on rapid procedures for measuring the strength and elastic properties of rocks *in situ* by means of duroscopes and a Schmidt hammer together with other appropriate tests of rocks *in situ* and in the laboratory. Unconfined compressive strength can be measured by using a Schmidt hammer, and comparing the values of rebound with the results of simple compression tests made in the laboratory. In order to transform the results of point tests on rocks *in situ* to the whole rock mass, research work has been carried out, e.g., by Voropinov (1971). This has made it possible to determine the effects of the physical state and deformation of the rock mass on its strength. The result is that the boundary of the Mohr envelope for the rock can be transformed into an envelope for the rock mass. The susceptibility of the mass to fracturing expressed in terms of the specific frequency of fractures (ω) has the greatest effect on this transformation.

It is appropriate to supplement the geological, geophysical and geomechanical description of tunnels by making a continuous photographic record (Adamus, 1974). The photographs provide an objective record of all geological features exposed in a tunnel. The photographs are particularly important for determining the specific frequency of fractures (ω) in individual parts of the mechanical profile.

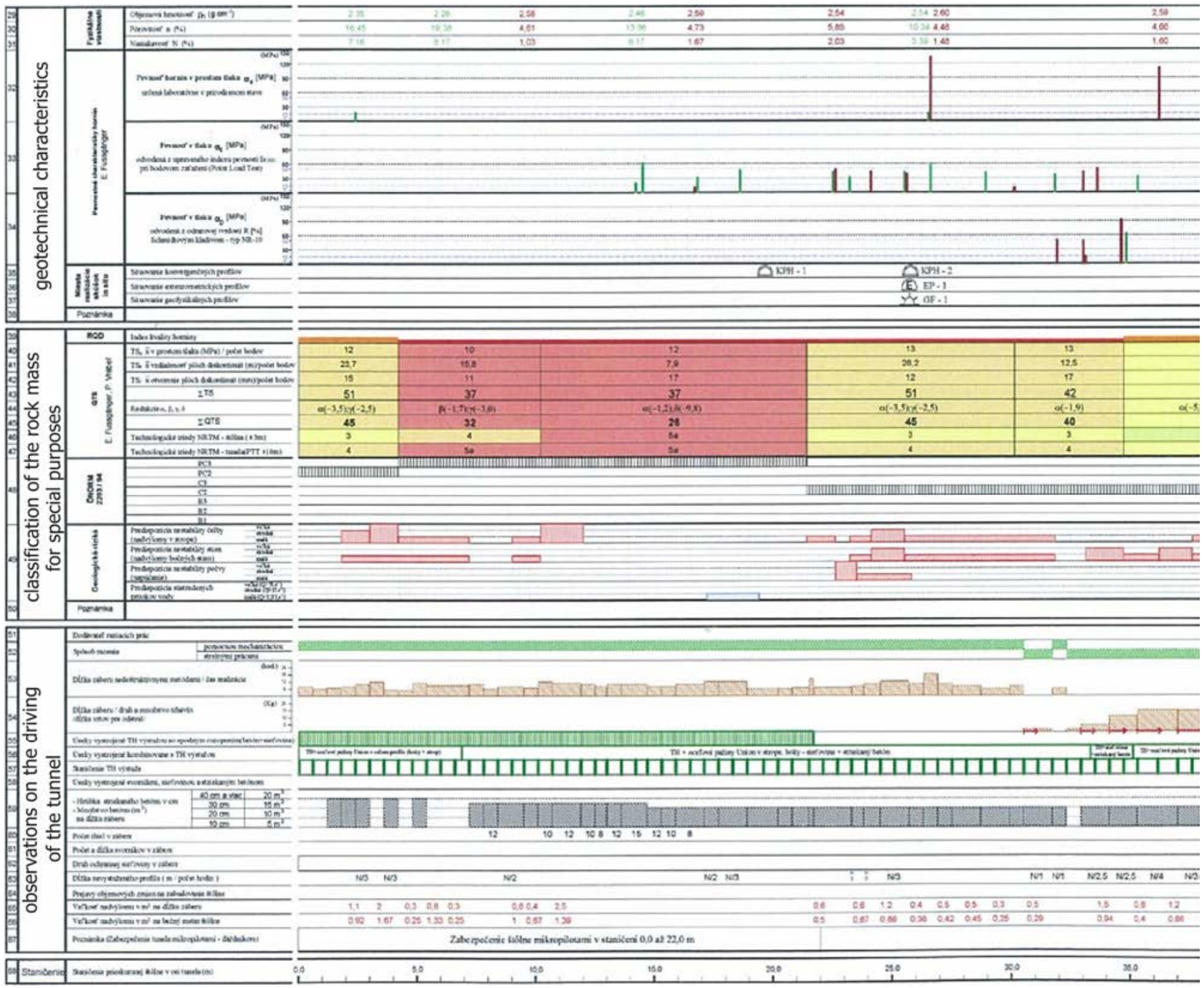
Figure 7.4.7 is an example of the photographs taken of exploratory tunnels at Hrhov. To enable subsequent evaluation at any time it is essential to use a system which records the exact location of the photograph within the tunnel. In the 1970s, when only black-and-white photographs were used, the main purpose of photographic documentation was to capture the character of the rock mass and the position of fractures in it. The upper photograph shows a thick sequence of Quaternary sediments through which a tunnel 52 metres long was driven. The Quaternary deposits at Hrhov were very heterogeneous, and as described above, their mechanical parameters in places reached values comparable to those of the bedrock. The lower photograph shows the karstic character of the massive Triassic limestone. Karst phenomena were minimal in that part of the studied mass and had no serious effect on the design of construction.

Fig. 7.4.8 Complete description of a tunnel (after Fussgänger, 2006)



In recent years, developments in digital technology have improved the quality of observations that can be made during a survey, and techniques have been introduced that were unthinkable in the past. This applies not only to engineering-geological, geotechnical and geophysical surveys, but also to the complete documentation of exploratory workings. The graphics packages now available allow all the observations and measurements made during the survey of an exploratory working to be displayed, regardless of the scientific procedure used to collect them. An example of such a complete record is the one made by the Slovak company GEOFOS shown in Figure 7.4.8 (Fussgänger, 2006).

Each category of information about the working is depicted in separate blocks of the diagram. In the illustration given, in addition to the basic data describing the tunnel (5 items), the engineering-geological and hydrogeological characteristics of the rock mass are shown (4 items), together with the geotechnical characteristics of the rock mass (4 items), classification of the rock mass for special purposes (5 items), and observations on the driving of the tunnel (17 items). The main blocks of documentation are sub-divided as follows:



- Engineering-geological and hydrogeological characteristics of the rock mass;
  - Geotechnical characteristics of the rock mass (9 sub-items);
  - Graphic depiction (3 sub-items);
  - Supplementary statement (4 sub-items);
  - Hydrogeological characteristics (7 sub-items);
- Geotechnical characteristics of the rock mass;
  - Physical properties (3 sub-items);
  - Strength characteristics of the rock (3 sub-items);
  - Locations of tests performed *in situ* (3 sub-items);
  - Remarks;
- Classification of the rock mass for special purposes;
  - RQD (1 sub-item);
  - QTS (8 sub-items);
  - ÖNORM (7 sub-items);
  - Geological hazards (4 sub-items); and
  - Remarks.

In an engineering-geological survey, it sometimes happens that it is not possible to correlate zones of weakness in one exploratory working with those in another, or to project zones from exploratory workings to the surface. In this case geophysical methods can be useful. It is possible to apply certain conventional geophysical methods or to use the mise-a-la-masse method. If one of the current electrodes is earthed in a fracture zone exposed in one tunnel, it is then possible to search for the corresponding fracture in another exploratory working or on the surface by making special measurements. Figure 7.4.9 shows the results obtained by using a group of conventional methods, including resistivity and magnetic profiling and measurements of magnetic susceptibility. A geological section was compiled by comparing the results obtained using these methods.

7.4.2 Description of Exploratory Boreholes

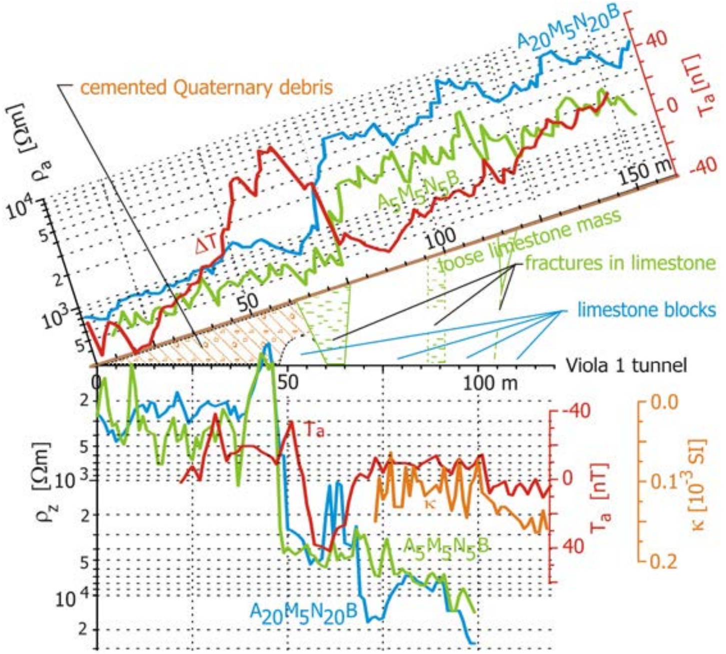


Fig. 7.4.9 Geophysical measurements in and above a tunnel

As in the case of tunnels, comprehensive descriptions are made of boreholes drilled using mechanical rigs from which core is recovered, and also of coreless boreholes. The latter, however, are not so suitable for the purposes of an engineering-geological survey. The geological logging of boreholes is based on the detailed description of the petrographic and lithological features of the rocks intersected in the hole, together with measurements of lineations and planar structures such as bedding, schistosity, foliation, faults and fractures. Core recovery is an important qualitative indicator of the mechanical coherence of rocks cut by the borehole. From the 1970s, measurements of the maximum length of core fragments were used as one of the criteria by which rock quality was assessed. A more objective measure of the influence of fracturing on rock quality is provided by using the so-called RQD index (Rock Quality Designation, Deere, Miller, 1966), in which the proportion of the total length of coherent core fragments longer than 10 cm is expressed as a percentage of the length of a measured section of a borehole. Because this parameter has been used successfully in all the surveys carried out by Geotest, certain useful modifications have been made, and a separate chapter of this book is devoted to the topic (Chap. 7.5). In addition to routine geological logging of boreholes, it is always helpful to carry out geotechnical and geophysical logging as well.

Geomechanical profiling can be carried out on cores recovered from boreholes using the same procedures used on the walls of tunnels and exploratory pits. The strength of drill cores is tested using portable field presses (Voropinov, *et al*., 1971). Using such tests, the unconfined compressive strength (σ_{md}) and the reduced strength (σ_{red}) can be measured, and the static modulus of elasticity (E) and the modulus of deformation (E_{def}) at the moment of rupture can be derived. A more suitable procedure for the mechanical description of exploratory boreholes is the method of mechanical logging (Zeman, Müller, 1973) based on the complete record of the rates of drilling, parameters of the drilling regime and the rate of wear and tear of individual drilling tools. Mechanical logging produces a continuous record of the reduced strength (σ_{red}), which is a comprehensive strength/deformation parameter characterizing the mechanical state of drilled rocks.

The observation and photography of the walls of boreholes by means of a borehole periscope (Šamalíková, *et a*l., 1974) is an equivalent of the photographic record made in tunnels. The drill periscope technique was superseded by the development of electronics and replaced by the use of television cameras. Images of boreholes produced by television cameras have already been illustrated in the preceding chapters. Nowadays it is possible to capture images in both axial and radial directions, and the latest techniques in processing televised data enable the wall of the whole borehole to be captured as a continuous image.

An illustration of the comprehensive documentation of borehole J 1003 from Dalešice is given in Figure 7.4.10. This borehole in the area of a future powerhouse was drilled in granulite using diamond bits. The excellent state of the core enabled detailed petrographic descriptions of the rocks and the determination of core recovery and the RQD index. The dips of foliation planes and joints and their properties were recorded. Faults and fracture zones in the granulite are characterized by brittle failure with minimum values of core recovery, RQD, apparent resistivity, and velocities of longitudinal seismic waves. Logging measurements were complemented by laboratory tests on selected core samples (bulk density, wave velocity, and coefficient of reflection). The measured values show a good correlation with the results of logging. The results of caliper logging show that there is also an increase in the borehole diameter over the intervals coinciding with faults. The characteristics of the fracture at 32 metres are interesting. It is a typical example of brittle failure, indicated by the minima in core recovery, RQD and seismic velocity, whereas the resistivity does not change over this interval. Brittle failure is accompanied by intense fracturing of the rock mass which is also indicated by the formation of cavities.

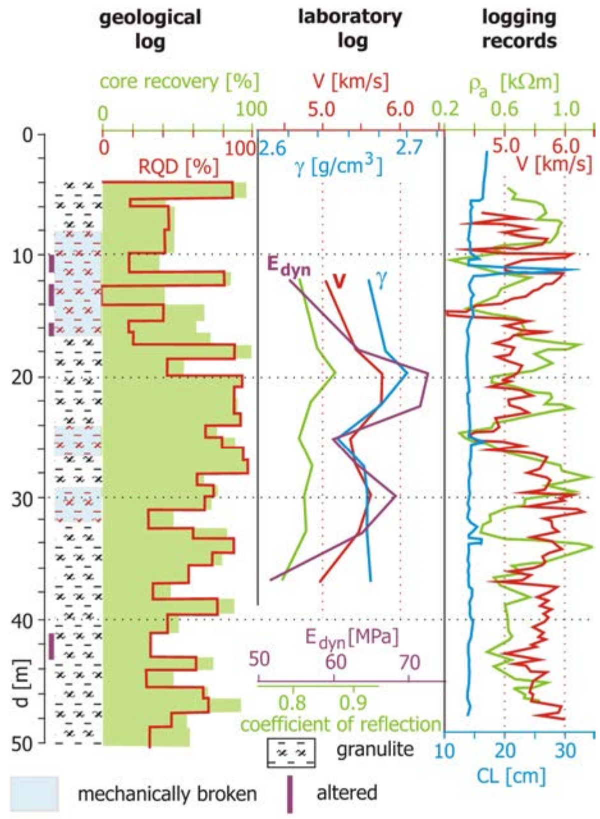


Fig. 7.4.10 Comprehensive logs of a borehole

In the 1970s, when probes for magnetic logging were not available, the opportunity was taken to make direct measurements of susceptibility on drill cores using a portable kappa meter. When evaluating these measurements, it was necessary to introduce correction coefficients to take account of the cylindrical shape of the core. An example of measurements made in a dam profile at Slezská Harta is shown in Figure 7.4.11. This technique was used to distinguish lava flows. The results were confirmed subsequently by palaeomagnetic measurements when differences in the size and direction of the vector of remanent magnetization in the separate lava flows were detected. Unusual measurements of this type are used occasionally to solve special problems.

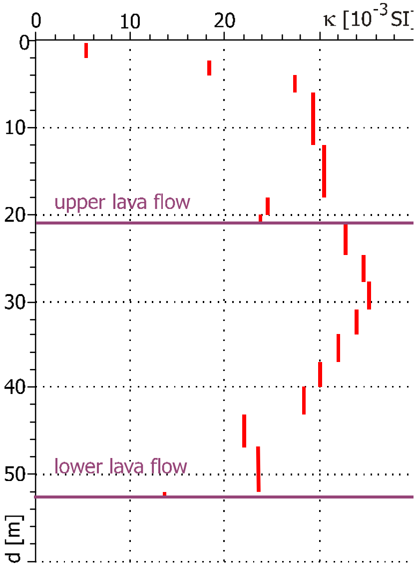


Fig. 7.4.11 Susceptibility of a drill core

Experience gained by carrying out engineering-geological surveys for large dam projects has shown that only comprehensive documentation of exploratory workings using the procedures described above will produce the range of objective data required for a correct interpretation of geological and geotechnical conditions.

The results of dynamic penetration can be expressed as the specific dynamic penetration resistance. Based on this, other important soil properties can be determined. When evaluating these, it is necessary to bear in mind that the SPT method used to determine these properties is indirect. In this way it is possible to determine consistency, relative density, cohesion, the effective angle of friction and, if need be, the modulus of deformation as well. Nowadays, devices are available which are connected to a computer using implanted semi-empirical formulae. These give the values of a range of different soil properties directly. The evaluation of tests is obviously not a simple matter because of the complex of factors governing the state of stress of soils. It is therefore recommended that certain penetration tests be made close to direct exploratory workings or sites where other geotechnical tests have been made so that the interpretation of the penetration tests can take account of the results obtained from other measurements. An example of the results of a dynamic penetration test is shown in Figure 7.4.12.

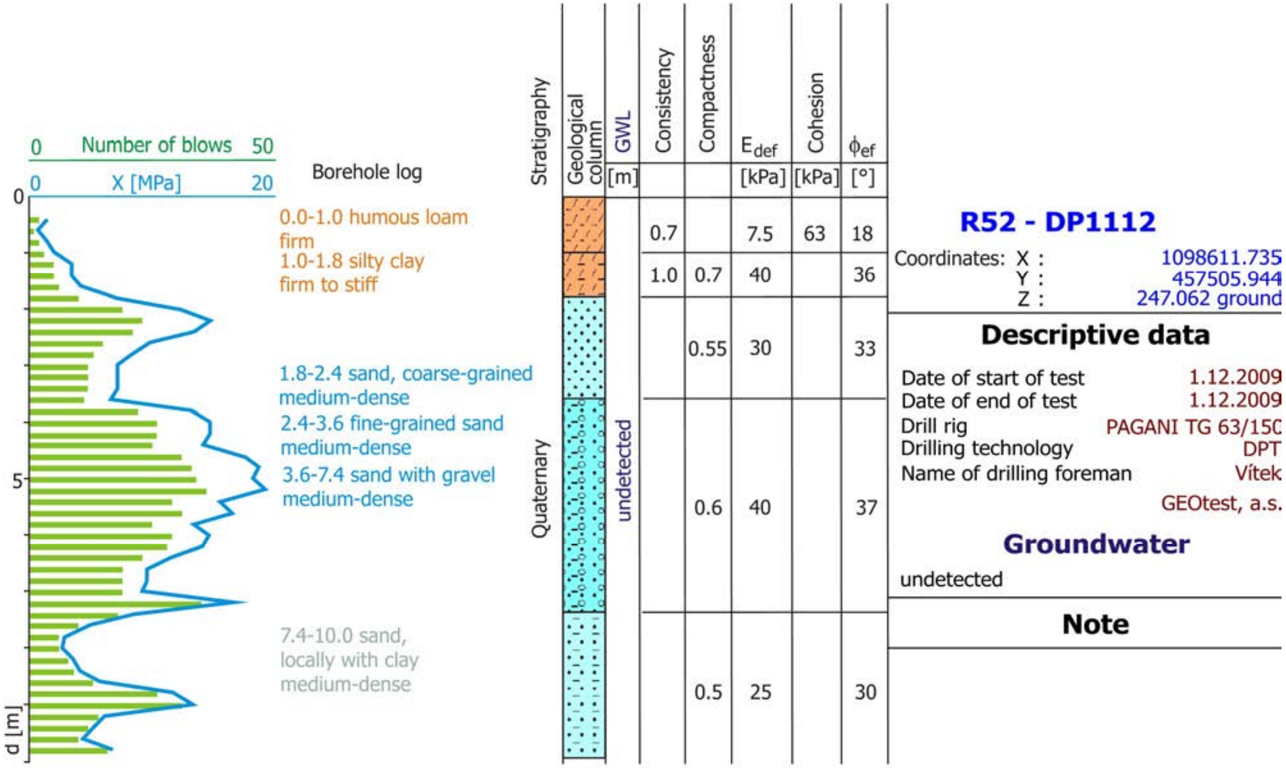


Fig. 7.4.12 Dynamic penetration test

When carrying out a conventional static penetration test (CPT), values for tip resistance and skin friction are measured with a load cell and displayed on an electronic readout. If possible, pore-water pressures are also measured. As in the case of dynamic penetration, modern equipment used for static penetration tests is computer-controlled and continuously gives the values of specific penetration resistance, the force on the conical tip and skin friction. The most advanced equipment enables a wide range of sophisticated measurements to be logged in penetration holes. Radioactivity, magnetic and electrical parameters are those most commonly measured. From these it is possible to determine the bulk density of soils, their water content and often their resistivity, magnetic susceptibility and exposure rate. The knowledge of these values, in combination with the mechanical properties measured using the static penetration method, enables the soil mass to be described more precisely. An example of such measurements is given in Figure 7.4.13.

7.4.3 Observations Made during Construction

A special part of survey work consists of the observations made during construction of the dam and ancillary facilities. These are extremely important to enable the predictions made as a result of the engineering-geological survey to be checked, and, if necessary, to make any modifications that are necessary for the safety and technical success of the project. When the footing at the bottom of the dam is exposed and tunnels and foundations are excavated, new discoveries are frequently made that call for small but essential change in the project and in the method of construction. Experience shows that in those cases where the engineering and construction of dams is challenging because of complicated topography and geology, continuous geological and geotechnical monitoring during the period of construction can prevent accidents and guarantee the successful outcome of the project. An experienced engineering geologist is able to make proposals for modifications to the bottom footing of the dam, especially in deciding the method of the excavation and its dimensions. Recommendations can be made concerning the maximum depth at which explosives should be used and at what point the rocks should be broken by hand. For example, during the construction of the pumped storage hydroelectric plant on the river at Dalešice, geological supervision was carried out over the whole period of construction and was focused especially on the description of the diversion tunnels, of the excavations and footing of the dam foundation, of the combined outlet and intake structure, of the excavations for penstocks, and of a stilling pool and an overfall, and of the foundation pit for a powerhouse. Also, grout holes and the construction of a grouting tunnel were continuously monitored. Geological observation of the cuttings excavated for access roads was continuous so that the composition of the rocks and soils in the cuttings would be available to the engineers of the

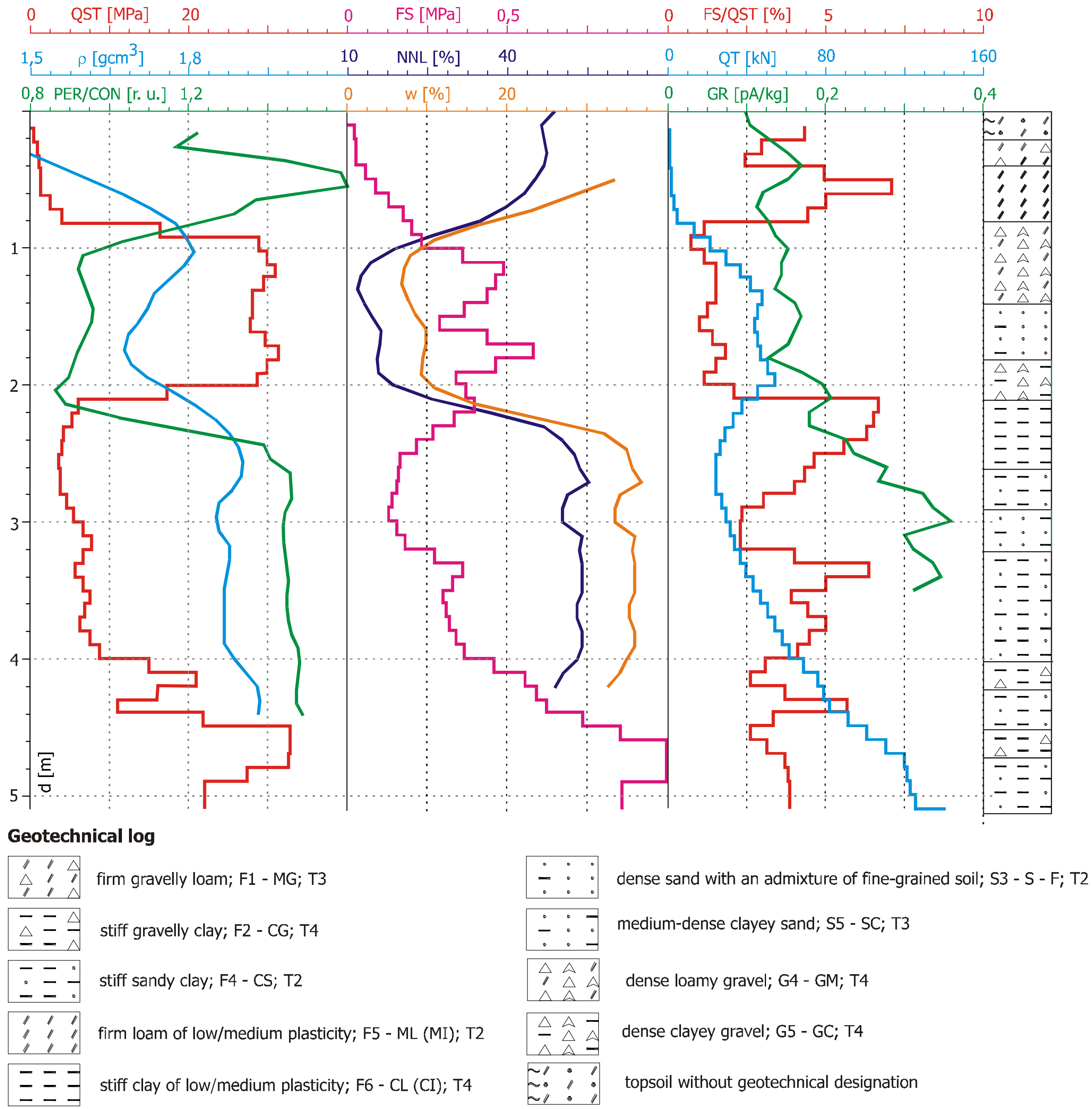


Fig. 7.4.13 Static penetration combined with logging (after Starý, 2010)

construction company. Due to the fact that the rate of progress of excavation work, both in tunnels and foundations and in the stone quarry supplying material for construction, was both fast and continuous, it was not possible to make observations and descriptions using traditional methods, so photography was used; the volume of excavations was measured by photogrammetry.

A photograph of the discharge from diversion tunnels at Dalešice is shown in Figure 7.4.14, and in Figure 7.4.15, there is a geological section based on the results of the geological survey and description of this excavation. As shown in Figure 7.4.15, the geological survey of diversion tunnel Št 2, made when it was driven in 1970–72, confirmed the accuracy of the geological interpretation made in 1970, by comprehensive evaluation of direct exploratory workings and of indirect survey work using a comprehensive range of geophysical methods.

Fig. 7.4.14 Diversion tunnels and channel at Dalešice (a photo by O. Horský - 1970)

When constructing dams, the geology of the exposed bottom footing must be described in detail. Because individual blocks are excavated successively, the geologist must visit the site regularly, or, when necessary, remain permanently on site to monitor progress during excavation and construction.

Figure 7.4.16 shows the results of a survey of the footing of the dam on the River Lubina near Kopřivnice. It incorporates a detailed petrographic description of the rocks, the measurements of the direction of dip of bedding, foliations and joint systems, and their characteristics, including the composition of their infill. Here, it must be emphasized that the proper cleaning of the footing bottom by water pressure is required so that observations and descriptions and photographs of high quality can be made.

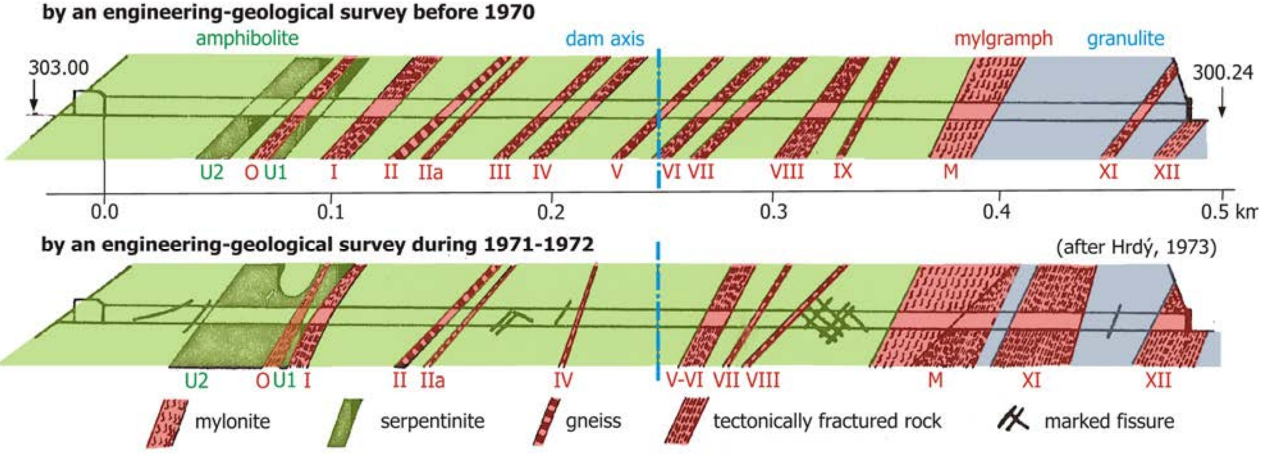


Fig. 7.4.15 Log of a diversion tunnel

An important part of the geological work at the stage of construction of a dam and its ancillary facilities is the technical supervision of the extraction and processing of construction materials and the classification of the rocks into categories of workability. An engineering geologist checks the compaction of earth and structural materials and ensures that samples are collected for checking in the laboratory so that, when necessary, loading and shear tests can be carried out. The engineering geologist and the geotechnician thus become an inseparable part of the team of specialists working through the entire period of construction and sometimes even after the construction is completed.

7.5 Application of RQD for Geotechnical Evaluation of the Rock Mass


Fig. 7.4.16 Observation and description of the base of a dam footing (a photo by O. Horský - 1972)

The current procedures used for describing rocks and the methods used for laboratory and field tests enable their geotechnical properties to be defined. In order for point measurements to be extended so that they provide a valid description of the properties of the rock mass as a whole, a range of direct and indirect survey methods are used which enable the rock mass to be divided into quasi-homogeneous blocks based on the correlation between the various measured parameters and to characterize these blocks in terms of the average geotechnical properties predicted. One of the readily available methods that is not used often enough is the determination of reduced core recovery – RQD (Rock Quality Designation – Deere, Miller 1966). Geotest systematically applied this parameter to the engineering-geological evaluation of the rock environment almost immediately after details of the procedure were published, so a separate chapter is devoted to this topic. Another reason for the compilation of this chapter is the critical evaluation of the method carried out by Geotest and the development of modifications that would make it more effective.

The index of quality, RQD, expresses the ratio of the cumulative length of drill core fragments more than 100 mm long as a percentage of the total length of a measured run of core. Reduced core recovery is indirectly dependent on the degree of weakening of the rock mass, i.e. on the degree of rock weathering, and the state of stress and fracturing, the abundance of joints and other factors affecting the coherence and isotropy of the rock environment. Another circumstance which may affect the value of RQD is the original state of stress in the rock mass. If the rock was exposed to high stress, the drill core has a tendency to crack and disintegrate soon after recovery. Therefore, if possible, it is necessary for measurements of RQD to be made continuously as drilling advances. If a core has been damaged during drilling or sampling, i.e. if fresh or regular fractures instead of natural fractures are observed, these lengths of the core can be measured as a single fragment.

It is important to note that in the former Czechoslovakia before the introduction of RQD (in the 1950s and 1960s), it was the practice to measure the longest fragments of core in individual runs of core when evaluating core recovery from boreholes. In this way a more objective assessment of the technical state of the rock could be made. This parameter had no designation and could lead to erroneous assumptions about the good state of rock if the fragment was derived from a block floating in a fracture zone or in the rock mantle. Once the RQD parameter was introduced, the procedure described above was no longer used.

When RQD measurements are made in sedimentary and metamorphic rocks that have abundant planes of discontinuity related to their deposition or deformation, it is necessary to proceed with more care than in the case of igneous and thick-bedded rocks. Sediments and low-grade metamorphic rocks generally weather faster and are readily affected by the climate. For these reasons it is necessary to measure their RQD as soon as the core is recovered from the borehole before any disintegration occurs. Generally, in these types of rocks the use of a bit with a double core barrel and a borehole diameter of at least 76 mm is recommended so that the core will be of maximum quality. It should be a matter of course for an engineering geologist to instruct the drilling foreman that not only is core recovery important, but also the quality and length of the core. The time required for drilling each core run, the rate of advance and the amount of thrust, and the colour and composition of the emerging drilling fluid are data which should be objectively entered in the drill record, because they are important when assessing the correlation between RQD and other geological parameters. In this case, mechanical logging is an advantage because all this information is automatically recorded.

Based on practical experience and correlations between RQD and other geotechnical parameters, a classification of the technical state of the rock has been developed. The most familiar is the classification given in the left section of Table 7.5.1.

Table 7.5.1: Evaluation of rocks using values of RQD and CRM

RQD (general use)		**CRM (local use – an example)**	
RQD	**Technical state of rock**	**CRM**	**Technical state of rock**
> 90 %	Excellent	> 90 %	Excellent
90–75 %	Good	> 50 %	Good
75–50 %	Average	50–25 %	Average
50–25 %	Poor	25–10 %	Poor
< 25 %	Very poor	< 10 %	Very poor

It must be emphasized that the classification given can be used provided that the required quality of drilling has been maintained and that core recovery is optimal. This can be achieved by using the correct type of drill rig and drill bits suitable for the rock being drilled, and by choosing appropriate lengths for the core runs, the optimum drill speed and amount of thrust. If optimum conditions are not achieved, the classifications by RQD cannot be used. If, for example, an unsuitable technique is used for drilling a given type of rock, it may be that core recovery from a single borehole or even over the whole area of a site will be low and the measured RQD will be very poor or negligible because core fragments rarely reach lengths of 10 cm. On the other hand, it might be discovered by using other objective methods that the state of the rock is relatively good. There are many instances when the technology used for drilling is unsuitable for the intended purpose but cannot be changed because of practical reasons. The parameter RQD cannot be applied in weak or unconsolidated rocks or soils.

Based on the evaluation of tens of drilled sites and a large amount of data, the conclusion has been reached that even when the technology used for drilling on a site is not optimum, it is still possible to reach satisfactory conclusions by comparing the information obtained by drilling with parameters measured by other procedures. In these cases, a modified classification of core recovery introduced by Geotest has proved to be useful. In this classification the length of the core fragments used to assess the rock quality is different from that used in assessing the conventional RQD index. In the engineering-geological survey for the thermal power plant at Santa Cruz, it turned out as to be useful to use a critical length of five centimetres to determine the modified parameter CRM for thinly bedded Miocene and Quaternary limestone and marlstone and the classification given on the right hand side of Table 7.5.1 is based on this procedure. Objective correlations between CRM and other parameters have also been established. This modification was successfully applied by Geotest to assess the technical state of rocks and for studying the fracturing of the rock mass at several sites. However, it was not used for the typological classification of rocks according to either the Q-system of Barton (1974) or the RMR (Rock Mass Rating) system in the sense of Bieniawski (1976).

From experience acquired in engineering-geological surveys for dams and related projects, it has been possible to establish correlations between RQD and certain other physical parameters. Based on detailed analysis carried out on the site of the Dalešice dam, it was possible to show that there is a correlation between RQD and the apparent resistivities logged in a number of boreholes (Tab. 7.5.2).

Table 7.5.2: Relationship between RQD and r_a in granulite at Dalešice

RQD	Apparent resistivity [Wm]	State of rock
85–100 %	> 1,000	Compact
50–85 %	600–1,000	Slightly jointed
25–50 %	330–600	Moderately jointed
10–25 %	170–330	Intensely jointed
0–10 %	Below 170	Crushed

This relationship is also observed in the results of surface resistivity measurements. The relationship of these two parameters was especially useful for defining the trend of faults and weakened zones in the rock mass. The relationship of both the parameters is also documented in the left part of the graph in Figure 7.5.1.

The right graph in Figure 7.5.1 shows a comparable relationship, in this case between reduced core recovery and the velocity of longitudinal seismic waves. Based on the correlations between the velocity of seismic waves and the dynamic modulus E_{dyn} and other moduli, it is possible to convert RQD into geotechnical parameters and thus to divide the rock mass according to the intensity of fracturing, and also to assign values of these parameters to specific parts of the rock mass. In this way it is possible to extend the validity of point measurements to the whole mass.

A good agreement between core recovery, RQD and apparent resistivity, caliper logging, the velocity of ultrasonic waves and the results of laboratory tests has already been illustrated in Figure 7.4.10. It would be possible to plot a large number of graphs which would demonstrate the existence of relationships between different measured parameters.

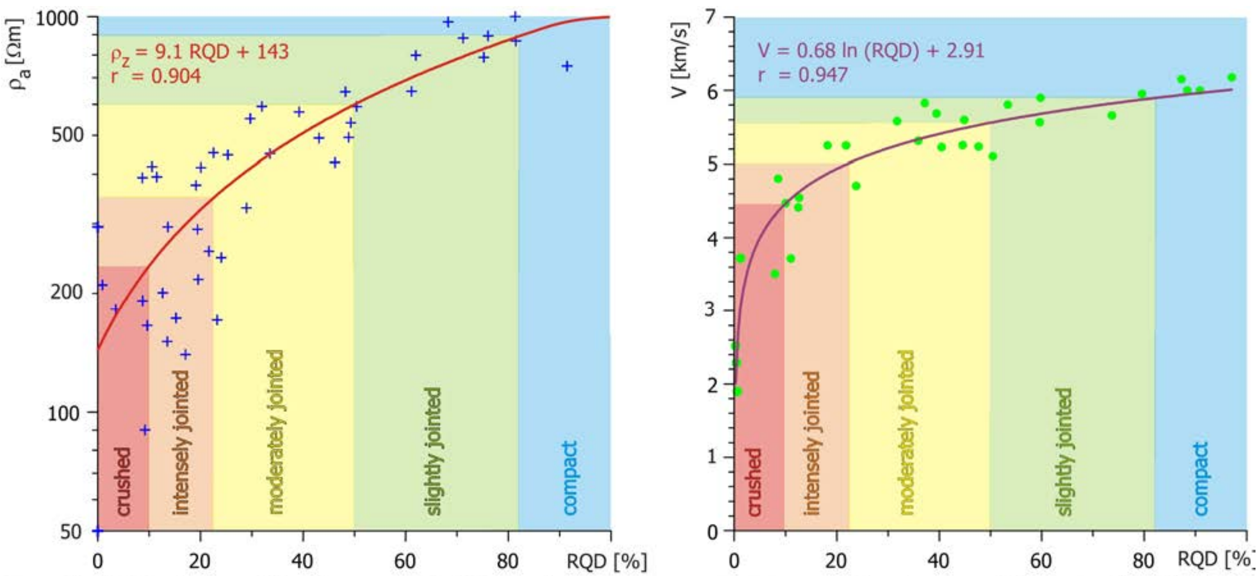


Fig. 7.5.1 Relationship between RQD and apparent resistivity and velocity of longitudinal waves

Strengths of samples measured in the laboratory are not always in good agreement with RQD and sometimes there is an inverse relationship in which high RQD correlates with low strength, or low RQD correlates with high strength. Such cases can be explained by the fact that in the laboratory it is usual practice to test intact samples of core. Such pieces can also be found in sections with low RQD. The RQD criterion used to assess the technical state of the rock will enable a correct interpretation of the data when anomalously high strengths are measured in the laboratory.

Figure 7.5.2 shows the correlation between RQD and the results of sonic logging, seismic radiography and specific water losses measured by water pressure tests. The Figure shows that the sections of the borehole with high RQD correspond in this case to the sections where specific water losses are minimal and the velocity of the longitudinal seismic waves increases. There is good agreement between the results obtained using different methods and on this basis the distribution of stress in the rock mass can be determined. It is of particular interest that the zone of concentrated stress caused by the high slopes of the valley and also the stresses due to the weight of the overburden can both be distinguished.

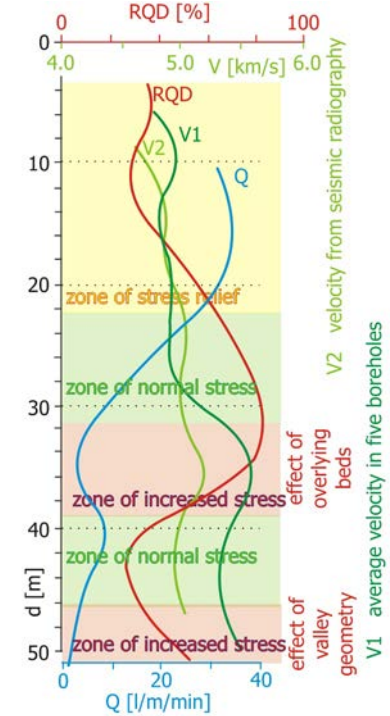


Fig. 7.5.2 RQD, WPT, and V

When comparing the measurements of seismic velocity obtained using different methods, the question always arises as to which of the absolute values of velocity are correct. There are cases in which higher velocities are measured when using seismic radiography and other cases in which higher velocities are measured using logging methods. One of these cases is illustrated in Figure 7.5.2. In this case, the explanation must be because cementation was used during drilling and water pressure tests. For assessing the properties of the rock mass it is better to use the velocities measured by seismic radiography. In this procedure, the seismic wave passes through the whole rock mass, not only through the rocks close to the borehole.

When making a survey for the thermal power plant at Santa Cruz, the application of CRM using a length of 5 cm for the critical length of the drill core fragments proved to be the simplest and most reliable criterion for delineating the boundary between Early Quaternary near-shore limestone and buried pre-Quaternary eluvia; for determining the boundary between rocks with different degrees of fracturing and the eluvial layer; for identifying cavities with infillings in limestone; for locating the karstified zone near the water table and areas where sea water intruded into the rock mass; and also for defining the position of faults.

Due to the possible occurrence of cavities, it was necessary to use a relatively closely spaced and regular network of exploratory holes in the survey for the thermal power plant at Santa Cruz. Because of this, it was possible to evaluate CRM in horizontal sections at different depths and define planar zones in which the technical state of the rock was different (Fig. 7.5.3). Thus, it was possible to identify weak zones in the rock mass at different depths and, by comparing the CRM data with the results obtained from other survey methods; it was also possible to trace the position of faults and karst zones.

The examples given above show that RQD and the modified CRM index are of great value in assessing the technical state of rock and also have wider applications in the definition of zones within the rock mass and for tracing the position of faults and weakened zones. It must be emphasized that the fullest interpretation of the results obtained using these two criteria of rock quality can be made only in combination with other survey methods. The assessment of their correlation with other parameters is a time-consuming procedure that depends on long experience and a thorough understanding of the engineering-geological and geotechnical conditions at a particular site. It is important to remember that every system of classification can be modified to suit the geological environment and technical circumstances in which it is applied, and according to the purpose for which it is intended.

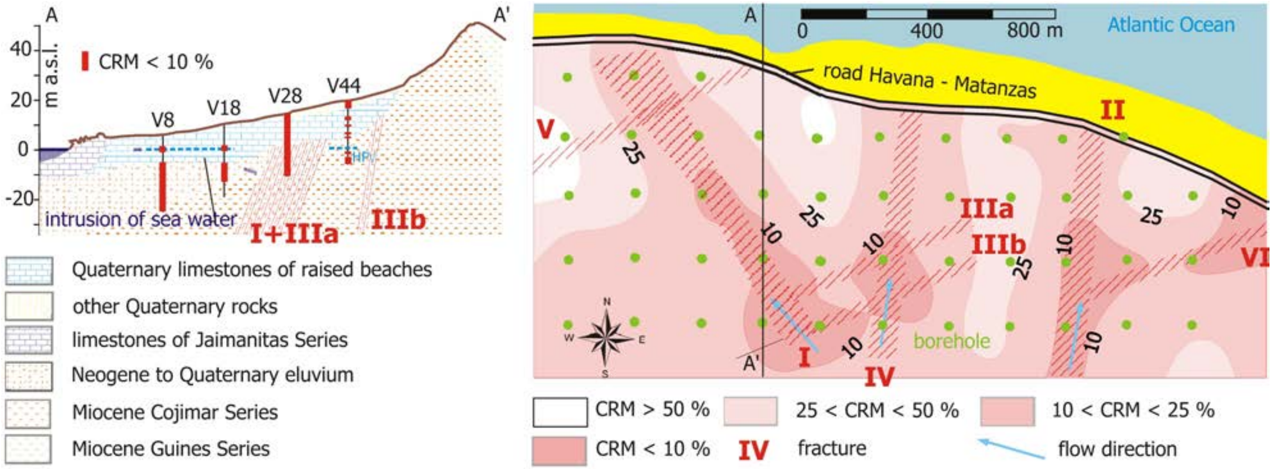


Fig. 7.5.3 CRM in a geological cross section and on a map of a construction site for a power plant

7.6 Trends of Direct Survey Work

The analysis of the possibilities of direct survey work shows that it is not always possible to guarantee the achievement of the required quality, particularly in drilling in which the maximum recovery of drill core is required to be ensured. Such a drill core should be unaffected by activities of other specialists before its description and the determination of RQD values. At the same time, it is necessary to bear in mind that even if drilling is carried out in high quality, another factor that may affect the result is the quality of description that will depend on the knowledge and experience of the geologist. The quality and objectivity of the final outputs, therefore, must be supported by a number of other exact measurements, down-hole logging, laboratory work, pressiometric tests in boreholes, etc.

In recent years, optical documentation of borehole walls by using a television probe has been applied more and more, being so much improved that it significantly contributes to the objectivization of results. The development (unfolding) of the image of borehole walls into a rectangle, when applying the ABI or OBI methods, enables us to document perfectly all planar and linear features such as bedding, foliation and the occurrence of joints and fractures in the close vicinity of the borehole. Their statistical processing, which is made possible by the software of new logging probes, provides information about the important features of the geological structure of the studied rock mass.

Modern probes for optical documentation of boreholes enable not only the inspection of borehole walls, but also the detection of the state of fracturing of the rock mass, the occurrence of cavities, water flow and the technical condition of the borehole. It can be said that a part of boreholes can also be made by full hole drilling where it is not necessary to collect samples for determining mechanical properties of rock, or if only limited financial resources are available for a survey, or for other reasons.

8 Geotechnical Surveys

The rapid development of methods used for calculations in engineering geology and geotechnics during the second half of the last century, particularly the application of finite elements in mathematical procedures used to model real systems, means that it is essential to have detailed knowledge about the physical state and tectonic structure of the rock mass and changes in its geotechnical properties in space and time within the area affected by construction work. Therefore, a geotechnical survey of a dam site must be designed to acquire the maximum qualitative and quantitative information about the properties and behaviour of the soils and solid rocks that underlie the site. To obtain this information, measurements are made both ***in situ*** in the field and on samples transferred to the laboratory, however the measurements made directly in the field provide the most realistic information about the behaviour of the rocks and soils. Only by the detailed investigation of the properties of the rock mass in the period before the start of construction of a dam can its behaviour be predicted during construction, after completion and when it is commissioned. It is crucial to understand the stress distribution and the likely extent of deformation caused by the mass of the dam itself, to assess the stability of slopes in the reservoir area, and to identify zones through which seepage could occur.

8.1 General Rules for Geotechnical Surveys

The comprehensive programme for a survey of a dam site includes engineering-geological mapping, the study of the topography of the site, the investigation of active geodynamic processes and their predicted effects, a geophysical survey, subsurface exploration work, and field and laboratory tests. The objectives of engineering-geological and geotechnical surveys are similar in many respects but the geotechnical survey is more concerned with the stability of the foundations of dam bodies and facilities, the permeability of the subsoil beneath dams, the load-bearing capacity of the rocks and soils, their deformation and shear characteristics, and the state of stress in the rock mass. It is also concerned with the technology and organization of construction, especially the assessment of workability, the use of excavated material for building purposes, the calculation of rates of inflow of groundwater into construction pits, their drainage and sealing, the measurement of chemical aggressivity of groundwater, the temporary stabilization of slopes, and the diversion of water during construction. The investment required for the type and technical design chosen for the hydraulic structure is also assessed in collaboration with the designer. Two of the important matters for discussion will be the provision of an adequate supply of construction materials and the geotechnical conditions in the reservoir area; these topics are discussed elsewhere in this book.

Whereas the engineering-geological survey is chiefly concerned with the geological, topographic and hydrogeological conditions of the site and identifying the geodynamic phenomena that are active or might become active during construction and after the hydraulic structure is commissioned, the geotechnical survey is concerned especially with the following:

- The quantification of the properties of individual rocks and soils necessary for the technical design;

- The determination of the usability of rocks and soils for construction purposes, measurement of their properties, and the design of a procedure for processing them;
- The procedures necessary to guarantee stability of slopes, especially the stability of the slopes of construction pits and stopes in underground excavations where support is required;
- The workability of soils and rocks; and
- Recommendation of the most suitable technology for construction in relation to the predicted behaviour of the natural environment.

The client is particularly interested in whether the site selected is suitable for the construction of a dam. For example, according to standard ČSN 75 2410 “Small Water Reservoirs”, the scope of the basic information required and the detail of the survey undertaken will depend on the extent of the documentation being prepared, the size and function of the reservoir and the complexity of natural conditions. This standard can also be applied to large water reservoirs. It is natural to expect that the scope of survey work will be enlarged to match the increased scale of construction required for a major hydraulic structure.

General rules apply to the procedure used in contracting for individual stages of the survey. These must be respected regardless of the stage that is the subject of the contract or request. These conditions, listed in order of importance, are as follows (Fousek, 1982):

- The contract should always be to carry out the survey as a whole and not for carrying out a certain type or number of exploratory workings, except in those cases in which it is necessary to reappoint a contractor for the survey, e.g., because of statutory provisions;
- All relevant information required to design the programme for the survey work must be made available (e.g., the dimensions and type of dam, the intended function of the facility, the assumed or preferred type of foundation, the intended scale and shape of excavations, warnings about special operating conditions and states of loading, etc.);
- The results of previous survey stages must always be provided as a basis for the next stage of work;
- Permission must be granted to enter the plots of land on which exploratory workings will be carried out and information about the locations of utility lines on them (water, electricity, gas, telecommunications) must be supplied by the administrators concerned;
- The contractor for individual stages of the survey should not be changed unless there are serious reasons, for example, legal requirements or decrees; if, for any reason, this procedure cannot be followed, special measures must be adopted; and
- Sufficient time must be allowed for carrying out and processing the results of the survey. Particularly at the stage of the detailed survey, time must be allowed for the progressive opening of exploratory workings and to make the necessary geotechnical measurements; the practical testing of some results will often contribute to the quality of the outputs.

When selecting the contractor for a survey, the contracting organization should fulfil the following requirements:

- Certified systems of quality management (ISO 9001) and environmental management (ISO 14001) should be in operation;
- Laboratories used for analysis of water chemistry and soil mechanics should be accredited;

- Those responsible for geotechnics should be authorized engineers of the Czech Chamber of Certified Engineers and Technicians, those responsible for engineering geology and hydrogeology should be certified as professionally competent by the Ministry of the Environment of the Czech Republic; and
- The contracting organization should have a record of successfully completed surveys for hydraulic structures.

The preparation of the design of an investment project takes place in stages in the same way as the survey. In them, the details of an investment project are specified in the same way as engineering-geological, hydrogeological and geotechnical information. The optimal sequence of stages and the instructions for contracting them are given in Table 8.1.1 (Fousek, 1982):

Table 8.1.1 The optimal sequence of stages of investigation

Orientation survey	**Purpose**	The basis for a feasibility study;
	Tasks	Fundamental assessment of the suitability of options chosen for siting a water structure in terms of engineering geology, hydrogeology and geotechnics;
	Necessary background	A prefeasibility study; if not available, then at least information about the alternative locations for the dam profile, the dam height, and the main functions of the intended water structure must be provided;
	Detailed requirements (DR)	• Essential information (mainly qualitative) about all options for a dam site, sites for extraction of construction materials, and the reservoir area: o Topography (ruggedness, gradients, manifestations of erosion action, signs of slope movements); o Geology (the genesis, composition and general thickness of the superficial deposits and the probable depth and lithology of the bedrock); o Hydrogeology (hydrogeological behaviour of individual layers or formations, the probable position of the groundwater table); o Special properties (seismicity, the presence of mineral deposits, wellfields, protection zones, reserves or protected areas, undermining, landfills, massive contamination); • Comparison of the suitability of all studied sites with respect to all the given parameters; • A general proposal for further surveys of each of the evaluated sites;
	Usual methods and techniques (UMT)	• Collation of literature and archive data; • Study of geological maps; • Reconnaissance of sites and their surroundings, including description of natural and artificial exposures, local watercourses, and wells. If archive and literature sources are lacking, then a minimal programme of exploratory workings (drilling and excavation) and measurements using indirect methods (geophysics) must be made;

<table>
<tr><td rowspan="5">Preliminary survey</td><td>Purpose</td><td>The documentation required to justify the granting of a planning permit;</td></tr>
<tr><td>Tasks</td><td>Confirmation of the suitability of the site selected for a water structure; assessment of the technical feasibility of the investment in terms of engineering geology, hydrogeology and geotechnics, particularly the siting of a dam and ancillary facilities, the design of foundations for the dam and facilities, the shape of the dam body, the resources of deposits of construction materials and the quality of soils and rocks in them, the stability of reservoir banks, potential leakage of dammed water, etc.; statement of the plan for solving geotechnical problems associated with the investment and the basic parameters of subsoil and soils in borrow pits;</td></tr>
<tr><td>Necessary background</td><td>The feasibility study;
The report on the orientation survey;</td></tr>
<tr><td>Detailed requirements</td><td>Characteristics of the rock environment along a selected dam profile and in the area of selected borrow pits in terms of:
• Geology: character and sequence of layers, their genesis, spatial arrangement (thicknesses, depths) and their variability, character of weathering and its extent, systems of discontinuities in the rock mass (if relevant for design), seismicity, karst phenomena or other cavities, undermining;
• Geomorphology: topographic characteristics, manifestations of slope movements, signs of erosion;
• Hydrogeology: hydrogeological behaviour of individual layers or formations, position of the groundwater level and its fluctuation, estimates of yield of existing groundwater bodies, directions of flow, existence of groundwater resources in areas of interest (dam, site of deposit of construction material, reservoir area) or in their proximity, their yield and exploitation, possible consequences, groundwater aggressivity;
• Geotechnics: basic physical and mechanical properties of the subsoil.
At this stage, it is necessary to assess the impact of the construction on the environment by preparing an EIA study and to determine what negative effects the external environment may have on the construction; it is convenient to carry out a single comprehensive assessment in which engineering-geological, hydrogeological and geotechnical issues are reviewed. Surveys of archaeology, historical buildings, and buildings of architectural importance, as well as surveys of biocoenoses and the identification of land ownership and external interests are also carried out during this stage;</td></tr>
<tr><td>Usual methods and techniques</td><td>• Application of the results of the orientation survey (if this was carried out in the distant past, then changes in circumstances will be noted and omissions corrected);
• Detailed reconnaissance and field measurements of geological and hydrogeological phenomena;
• Subsurface exploration work combined with indirect methods;
• Laboratory tests;
• Preliminary geotechnical calculations (load-bearing capacity, subsidence, stability analyses);</td></tr>
</table>

	Purpose	Preparation of documents necessary for the issue of a building permit and of plans for construction work.
Detailed survey	***Tasks***	To provide detailed information for: • Safe and efficient planning of the project in relation to the geotechnical conditions at the site; • Selection of optimal technologies for construction; • Minimization of the impact of the project on the surrounding environment and neighbouring objects; • Identification of geotechnical risks; • A proposal for monitoring the construction. To specify the characteristics of the rock environment obtained during the preliminary survey;
	Necessary background (NB)	Documentation for issuing a planning permit, including static calculations; Report on the preliminary survey;
	Detailed requirements	Detailed determination of: • Mode of deposition of soils, the weakness of the rock mass and the position of the groundwater level; • Characteristic values of physical and mechanical properties of subsoils and dam materials for geotechnical calculations (limits of load-bearing capacity and deformation for ancillary constructions, limits of stability, deformation, and surface and internal erosion for the dam body); • Usability of local materials (volumes, bulking, parameters of compactibility, technology used for processing); • Plans for the use of special technologies (grouting, anchorage, reinforcement of soils), if considered necessary; • Plans for the design of drainage of the construction pit, if necessary; • Identification of geotechnical risks (definition of an acceptable risk, warning conditions, a proposal for preventive measures); • The plans for monitoring the construction (measuring the amount and rate of deformation of objects during construction, and of the surrounding objects, a proposal for the methods to be used and the points at which measurements will be made, and for the frequency and duration of measurements); measurements of changes in the groundwater level.
	Usual methods and techniques	• Application of the results of the preliminary survey; • Subsurface exploration work combined with indirect methods; • Field tests and measurements; • Laboratory tests; • Mathematical modelling;

Additional survey	**Purpose**	To make necessary changes in the procedure of construction work;
	Tasks	To supplement the results of the detailed survey, taking account of essential changes in the technology used for construction or changes in procedure caused by geotechnical problems or when marked deviations from the inferred geological structure or inferred properties of the subsoil at the dam site or in borrow pits are encountered;
	NB	A proposal for a change in the procedure of construction work;
	DR	Decisions are made by mutual agreement between the designer and the engineer responsible for the survey;
	UMT	The survey is carried out when, and if, circumstances require, and decisions are made so that the project can proceed with minimum delay;
Construction monitoring	***Purpose***	To ensure the smooth progress of construction and the subsequent trouble-free function of the dam and ancillary facilities, taking into account the real geological and geotechnical conditions; recording the geology at the site as construction proceeds;
	Tasks	Checking the accuracy of results and the specifications of survey activities; Monitoring the response of subsoil and the vicinity of the construction to the process of construction, and predicting their development; Monitoring the behaviour of objects and earthworks being built; Proposing measures for maintaining the responses of the subsoil within limits acceptable for the smooth progress of construction and the trouble-free function of the objects being built; Monitoring changes in the banks of the reservoir area; Monitoring changes and developments of the hydrogeological conditions in the vicinity of the project;
	Necessary background	Reports on geological and geotechnical surveys; description of the construction project; geotechnical specifications used for the design of objects;
	Detailed requirements	Decisions are made by mutual agreement between all participating parties (client, designer, contractors of works, engineer responsible for the survey).
	Usual methods and techniques	A programme of observations and measurements is designed to monitor the conditions at the construction site; the frequency with which various measurements should be made is decided and protocols for the continuous evaluation of results are established so that potential failures or dangerous circumstances can be detected. Depending on the circumstances, the following types of measurements will be made: Geodetic measurements of different types of deformation (subsidence, slip on normal faults, tilts); Measurements of forces and stresses; Measurements of the position of the groundwater level and pore pressures; Measurements of geoacoustic emissions; Measurements of consumption of grout mixtures.

If it is necessary to change the contractors responsible for different stages of the survey work carried out during a project, it is essential to prepare a proposal for each stage as required. These proposals should be drafted by the same person who will prepare an appropriate plan of survey work taking into account the results obtained during the preceding stage. This will be a definite proposal for the types and numbers of exploratory workings, tests, and measurements that should be made. This plan defines the tasks that will be undertaken.

It must be noted that if the team responsible for a survey is changed, the work flow tends to decrease and, as might be expected, the cost of the survey increases. The new contracting organization and the team of specialists appointed to carry out the survey must become familiar with all the technical details of the project and the geological environment in which it is being carried out. This means that they must be perfectly acquainted with the site and with all the documents containing the results obtained during preceding stages of the survey. This will be time-consuming and lead to additional expense for the client.

If a single contracting organization is made responsible for all stages of an engineering-geological and geotechnical survey, a consistent plan of work can be designed and the knowledge of the team of specialists evolves as the project progresses. This strategy is obviously beneficial in the case of large-scale projects carried out in remote or geologically complicated areas. It is also important to be aware that reports compiled at various stages of a project do not contain all the observations that have been made, and that there are always some features of the geology that cannot be given an unambiguous interpretation. All geologists understand this, but these unresolved problems will be kept in mind by a specialist team that is continuously involved at all stages of a survey. If the team changes, this continuity is lost.

The compilation of the information acquired at all stages of an engineering-geological survey should be clear and easily understandable and must be completed in accord with the schedule set by the client and the designer. Continuous consultation between all interested parties should take place. In the case of surveys for large projects, such that at Dalešice, it is appropriate to designate days for inspection of the construction site when leading experts from universities, academic institutions and other specialist organizations can participate.

8.2 The Rock Environment as a Determining Factor in Dam Design

Current developments in science and technology enable the construction of dams even in very complicated geological and geotechnical conditions. For a project to be successful, however, the function of the proposed dam and the associated buildings and services must be considered in relation to the type and extent of work required to construct them and the natural environment in which that work will be carried out. The fundamental factors that govern the design and the methods used in the construction of a dam are the topography and the composition and structure of the rocks on which the dam will stand. For this reason, despite all the advances that have been made in design and construction techniques, a complete knowledge of the rock environment is essential. A thorough engineering-geological survey of the construction site and the surrounding area must be made so that the effects of the planned project on the natural environment and its ecology can be properly assessed. The impacts caused by construction work may progressively lead to irreversible ecological changes that can also affect the rock environment itself, for example, by altering weathering processes and disturbing the groundwater regime.

If the information provided by an engineering-geological survey is inadequate, unexpected changes in the plans for construction may be necessary. In other cases, construction work at a site may be hindered by unexpected slope failures, uneven subsidence below the dam or under other structures being built, etc. In extreme circumstances, lack of geological or geotechnical information could result in the disastrous failure of a dam during construction or, after completion, when it is commissioned. In such cases there may be casualties and other grave consequences for the local community and the surrounding infrastructure, as happened at the Vajont dam in the Italian Alps (Figs. 2.3.31 and 2.3.32). Today, dams with more sophisticated structures are being built in more challenging geological situations so that a proper understanding of the rock environment and its geotechnical parameters is essential. The role of engineering geologists, geotechnicians and geophysicists in providing this information to guide the design and construction of a dam is more important than ever.

The Earth's crust is formed of rock. A rock is an aggregate of mineral particles and intergranular matter, held together by the cohesion of phase and grain boundaries and by forces of molecular attraction. The separate mineral phases possess specific physical and chemical properties and the physical characteristics of the rock as a whole are an expression of the properties and proportions of the constituent phases and the geometry of their boundaries. The variations in the physical and mechanical properties of rocks are thus governed by their mineralogical composition and texture. These characteristics are determined by the geological process responsible for the formation of the original rock and the changes to which it has been subjected after it was formed. Rocks with the same mineral composition can have quite different mechanical properties depending on their history of metamorphism and deformation and on the extent to which they are weathered. Generally, in nature, rocks are rarely found in a pristine condition. The Earth's crust is a dynamic environment in which conditions are continuously changing. This is especially true of the rocks near the surface that are exposed to the effects of stress release, weathering and erosion. This is the reality that must be faced when carrying out civil engineering investigations of dam sites and the geotechnical monitoring of construction work.

The measurement of the mechanical properties of solid rocks is one of the basic tasks of rock mechanics, but it is also a difficult one. This is because solid rocks, as compared to soils, are often distinctly inhomogeneous and anisotropic because of the geological processes that governed their formation and the changes that have affected them subsequently. All procedures used to determine the physical characteristics of a body of material, in this case a geological body or a quasi-homogeneous block of rock, depend on knowledge of the internal structure and the degree of homogeneity of the studied mass. If the internal structure of a rock mass is shown to be the same in all directions, i.e. the geometry of phases and grains is statistically uniform in all directions and the component phases are uniformly mixed, then the mass can be described as isotropic. In mechanical terms, the laws of elastic half-space are applicable to such a mass.

In the natural environment, homogeneous and physically isotropic rocks are almost never encountered. The structure and internal texture of the rock mass varies according to direction and its physical behaviour is therefore anisotropic. The anisotropy is mostly caused by planes of mechanical discontinuity (bedding planes, joint systems, planes of schistosity, faults and zones of dislocation, etc.), which disrupt the rock environment. If general predictions about the behaviour of the rock mass are to be made, it is necessary to take into consideration its inhomogeneity and anisotropy and the simple application of the laws of elastic half-space is not possible. For this reason, even when interpreting the results of tests it is necessary to carry out a statistical study of the discontinuities in the rock environment and to select and make tests, the results of which can be applied as widely as possible in the studied area. Therefore, it is important to understand that when examining coherent samples of rock in the laboratory, the measured properties will be determined chiefly by their mineralogy and texture in accord with the dimensions of the sample, whereas tests carried out on larger volumes of rock *in situ* will give results that tend to reflect the structural features of the rock mass as a whole.

New design techniques employing the finite element method and methods for modelling processes using real physical or mathematical models require the qualitative and quantitative parameters at all points in the rock mass to be defined. For this purpose, the goal of the engineering geologists, geotechnicians, geophysicists and other specialists (hydrogeologists, petrographers, mineralogists, geodesists, etc.) is to divide the rock mass into quasi-homogeneous blocks, i.e. into blocks that show the same mechanical behaviour throughout at a statistical scale of observation. Inhomogeneities and anisotropic behaviour in rocks of the same lithological type are caused by:

- Primary factors governing the formation of the rocks – differentiation of magma, sedimentary processes, texture, bedding, tectonic foliation, etc.; and
- Secondary factors:
  - Non-diastrophic (contraction and dilatation due to physical-chemical changes in the rock mass, exogene processes, etc.);
  - Diastrophic (manifestations of the stresses imposed on the rock mass – folds, joints, faults).

Four types of rock mass can be identified on the basis of primary and secondary factors that govern their mechanical inhomogeneity and anisotropy:

- Those in which the inhomogeneity and anisotropy of the rock mass is due primarily to internal differences in mechanical properties;
- Those in which inhomogeneities of the rock mass are governed by primary as well as secondary (mainly non-diastrophic) factors; the mass is penetrated by planes of mechanical weakness, on which there are qualitative and quantitative differences in strength;
- Those in which the inhomogeneities in the rock mass are caused mainly by secondary (diastrophic) factors, resulting in discontinuities; the mechanical properties of the rock mass then depend on the frequency of such discontinuities and the character of their surfaces and infilling; and
- Those in which the rock mass is cut by distinct fractures along which definite movements have occurred; these dislocations (faults) have different mechanical properties and mostly form the boundaries between quasi-homogeneous blocks.

All of the above categories of rock mass may be encountered in the process of surveying a dam site and designing a dam. The task of the engineering geologist is to divide the mass into quasi-homogeneous blocks that will chiefly be delineated by boundaries between lithologically distinct units and by persistent dislocations. They blocks must then be assessed in terms of their internal variations, namely:

- The distribution of less marked planes of mechanical discontinuity, the continuity of which was only generally observed when the quasi-homogeneous block was delineated;

- The distribution of joint systems which govern the behaviour of the quasi-homogeneous block; such joint systems must be classified according to their relative age and orientation, their frequency, continuity, the morphology of their surfaces and their infillings, and their openness (separation of their surfaces), etc.;
- Shear tests must be carried out to determine the shear strengths of rock blocks and planes of mechanical discontinuity; and
- The response of the block to stress must be determined by making loading, deformation, seismic and other tests; geophysical, particularly seismic, measurements enable results of point tests to be extended to the whole block within a quasi-homogeneous block.

The state of stress in the rock mass at a dam site prior to the start of construction must also be determined in relation to the spatial distribution of permeability. This is necessary so that changes taking place in space and time during construction of the dam and after it is commissioned can be estimated.

8.3 Methods Used for Geotechnical Surveys

Of special importance are the tests carried out on the rock mass *in situ* under natural conditions and laboratory tests carried out on rock samples, occasionally supplemented by the investigation of discontinuities created artificially. Although these tests are repeated in order to obtain data that can be processed statistically, the results are always a simplification of the real conditions in the rock mass. Generally, it can be said that the procedures now used to measure the deformation characteristics and the strength of the rock, and to measure the state of stress in the rock mass, and other properties and discontinuities of the rock mass, are so sophisticated that the computation of the respective parameter does not depend only on the statistical evaluation of a large number of tests. The correct transformation of the interpretated values so that they can be applied to the rock mass as a whole must be based on a thorough knowledge of the geological development of the area of interest and made in direct collaboration with the engineering geologist, the geotechnician and the geophysicist. An important part of the geotechnical survey should be to evaluate the effects of active geological processes on buildings that have already been constructed in the area of interest. This will provide a preliminary idea about the possible behaviour of the future dam during construction and after it is commissioned.

Laboratory tests are carried out to determine the physical, mechanical and technological properties of soils and rocks. In the case of soils, they include specific measurements of grain-size distribution and limits of plasticity according to Atterberg, measurements of physical properties and states (specific and bulk density, natural moisture, degree of saturation, consistency, swelling capacity, contractibility, etc.), determination of strength parameters (box shear tests, axial and triaxial tests in different configurations), tests of deformation characteristics, tests of permeability and technological tests (Proctor test, determination of CBR). The collection of samples and their testing in the laboratory form the basis for the description and classification of rocks. Common laboratory measurements made on solid rocks include especially the determination of the physical properties of the rock, tests of compressive and tensile strength, and technological tests (coefficient of softening, Los Angeles abrasion test /crushing test/, abrasiveness).

If necessary, petrographic descriptions are also made. They are necessary to determine the mineralogical composition and internal structure of the individual units that compose the rock mass. Elasticity and deformation characteristics are also measured in the laboratory using both static and dynamic tests. The compressive strength of rocks is measured by making simple pressure tests, and by pressure tests using coaxial dies, the tensile strength is measured using simple tension tests by stiffening the sides of a sample with epoxy resin and clamping it into jaws and by tension tests under bending; shear strength is also measured (Fig. 8.3.1). It is desirable to carry out tests not only in uniaxial, but also in triaxial mode, particularly at the more advanced stages of a survey. Measurements of permeability are also made on samples in the laboratory. All these tests should be carried out in a certified laboratory.

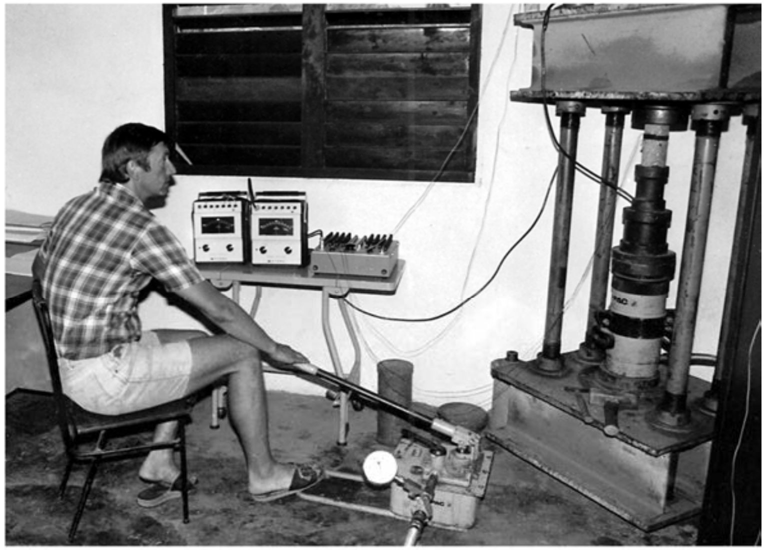


Fig. 8.3.1 Laboratory tests on rock samples

Once the measurements have been made and the results assessed, these can be used to constrain the design of the dam and the methods used for its construction. The engineering geologist and geotechnician must be aware, however, that properties or parameters of soils and especially rocks measured under laboratory conditions correspond more or less to those of the intact rock and are governed chiefly by the mineralogy and texture. They are therefore essentially point tests that are not representative of the rock mass as a whole that quite commonly contains numerous planes of discontinuity. These discontinuities markedly influence the behaviour of the rock mass so that the rock mass as a whole will have deformation and strength characteristics quite different to those measured in the laboratory. In order to overcome this problem, tests and measurements on larger areas or blocks of rock with larger volumes are made directly in the field. The scope of such field tests depends primarily on the design of the dam and on the complexity of the geological structure of the area under investigation. The tests are very demanding in terms of time and cost, therefore they are used mostly in cases where the design and construction of a dam and its facilities is complex and of large scale, e.g., pumped storage hydroelectric plants.

Laboratory tests of water include particularly full chemical analyses, partial chemical analyses for building purposes, bacteriological analyses, determination of the degree and character of contamination, and determination of the degree of radioactivity.

In the text below, only a brief list of basic field tests and measurements used in the geotechnical survey for hydraulic structures is given.

8.3.1 Identification of Deformation Properties

To measure the deformation properties of rocks *in situ*, the following field tests are used:

- The conventional loading test of a soil body or a rock mass is carried out by the application of uniaxial compressive stress. The force is usually produced by a system of portable hydraulic cylinders and is translated onto the subsoil through a bearing plate. The deformation of the rock is measured by using dial gauges or other sensors. The test is carried out on an open area and the stress is applied in a vertical direction; during this it is necessary to solve the problem of the counter-load in order to determine the load force (Fig. 8.3.2). For example, when carrying out loading tests on the compacted base course at the Centro Cuba site, a loading bridge consisting of concrete slabs was used as a counter-load. Similarly, conventional loading tests can also be made on the bottom of a pit or in a sufficiently wide borehole. In underground exploratory workings (tunnels, cross-cuts), advantage can be taken of the fact that rocks on opposite faces of a working can be tested by using one loading system either in a horizontal or vertical direction or in a selected oblique direction.

Fig. 8.3.2 Loading bridge being used to make a deformation test in the field (a photo by O. Horský - 1987)

- Large-dimension loading tests include, for example, water pressure tests formerly used in sealed sections of tunnels that were later replaced by loading tests using radially arranged presses (Austrian TIWAG or American USBR). The radial stress test is used primarily for detecting deformation caused by pressure in tunnels with circular cross-sections so that the correct dimensions for the lining can be determined. Radial stress is imposed on a test section by using either water pressure or a system of flat cushion presses. The result of the test is a graph showing the deformation of the rock mass forming the wall of the tunnel in relation to the imposed stress. This can be used to calculate either the average modulus of deformation or partial moduli in various directions.
- Large-area expansion loading tests with a loading area of 1.5 to 1.8 m^2 and with continuous measurement of deformation (static loading tests by a plate – Fig. 8.3.3).
- Tests carried out using deformometric probes in boreholes (elastometer, dilatometer, pressiometer, uniaxial press). These are essentially tests applied to the wall of a borehole by using radial stress and the deformation characteristics of the mass are

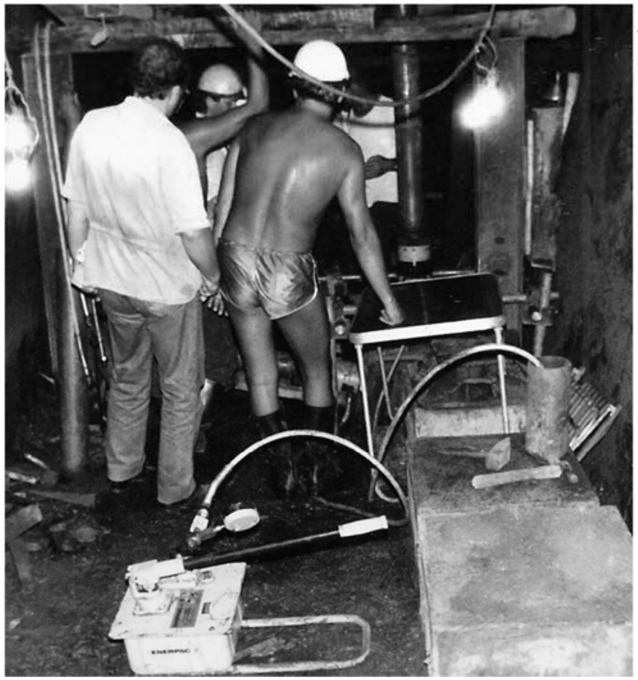


Fig. 8.3.3 Geotechnical tests in a tunnel, right – detail of sensors (photos by O. Horský - 1986)

determined by the expansion of the diameter of the borehole. The advantages of deformometric probes are their ease of deployment and their ability to measure the deformation properties of rocks at depth.

- Tests using flat presses in narrow trenches. In tests made using flat presses the deformation properties of rocks are measured by inserting a flat press in a slot drilled or cut in the rock. After the start of the test, compressive deformation of the rock in the vicinity of the press is measured (Fig. 8.3.4). Flat presses are hydraulic tools for the imposition of high specific loads. They consist of two plates connected along their perimeters, between which a fluid is injected under pressure through a filling hole. The test can be used to measure deformation characteristics in various directions, stress distribution around the walls of an underground working, and Poisson's ratio.

Fig. 8.3.4 Preparation of a test with flat presses (a photo by O. Horský - 1987)

8.3.2 Detection of Strength Parameters

By making field measurements of strength parameters, the strength of the rock as a whole can be determined as well as the strength along planes of mechanical discontinuity in the rock mass. The main tests carried out ***in situ*** are as follows:

- Tests under uniaxial compressive stress, the aim of which is to determine the stress at which a block of rock breaks up. This test is usually carried out in conjunction with a loading test in which, after deformation of the rock under uniaxial stress has been measured; the loading of the block is continued until it breaks up. The dimensions of the block depend on the type of rock and the equipment used for the test; as a rule, the lengths of the edge of the block range between 30 and 100 centimetres. The shape is usually a cube or a flat prism; the pressure is applied through the lower surface of the block.
- Shear tests combined with compression are used to determine the shear strength along a predetermined plane of failure (Fig. 8.3.5), which

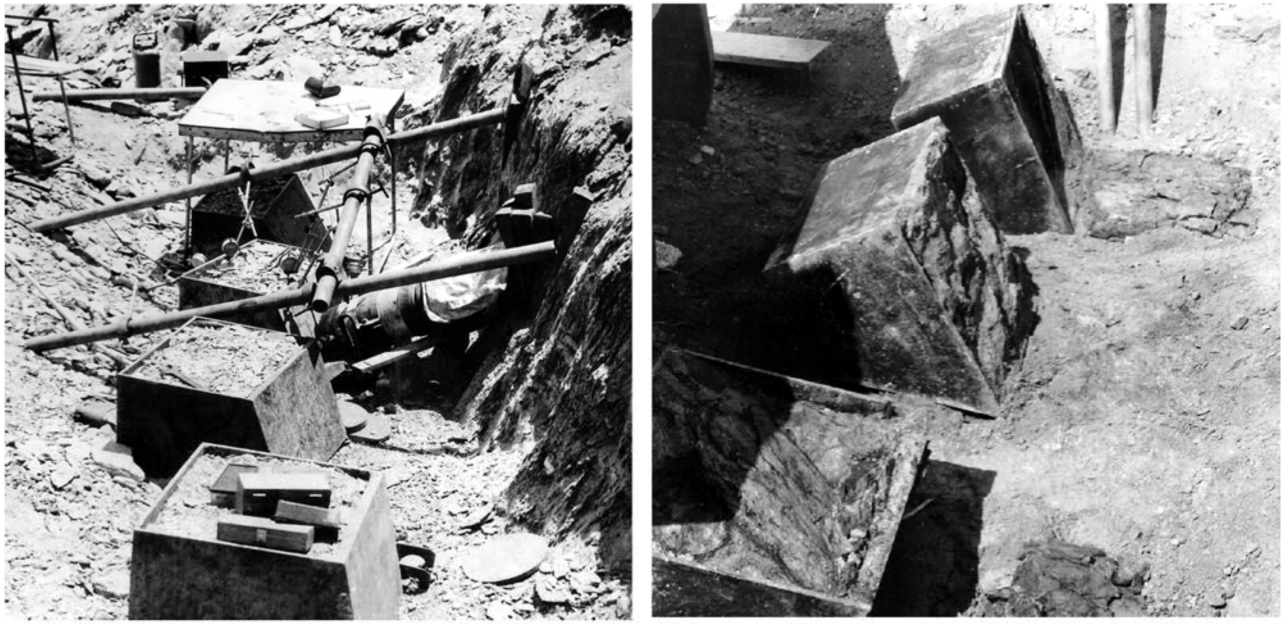


Fig. 8.3.5 Shear test in a frame: state before and after the test (photos by O. Horský - 1986)

is then the maximum tangential force applied to the plane of failure using a conventional arrangement of the test. The testing equipment must be positioned in relation to the plane of failure during these tests. There are a number of arrangements that can be used for these tests. They can be grouped as follows:

  - o Shear tests on blocks of rock not separate from the main mass;
  - o Shear tests on planes of mechanical discontinuity (they are carried out by moving two separate blocks of rock against one another); and
  - o Tests made by dragging concrete blocks across the rock.

- Corner shear tests are made by arranging the equipment on a block of rock loosened from the mass using two planes perpendicular to each other. The plane of shear failure is self-formed inside the mass. The arrangement of the test is similar to the triaxial test made in the laboratory. It provides a measurement of the limiting stress at which failure of the rock occurs so that the Mohr stress circle can be established.

Field tests are also used for measuring the strength parameters of soils. In comparison to laboratory measurements, they have the advantage of speed and it is not necessary to collect undisturbed soil samples. The commonest tests are:

- The vane shear test is used to measure the shear strengths of loose rocks. A device consisting of crossed planes (wings) is pushed into the soil and torque is applied using a lever; the rotating vanes cause the soil to shear around a cylindrical surface. The test determines the resistance of a soil to shearing, i.e. its cohesion. And

- The dynamic penetration test named the “Standard Penetration Test” (SPT) is used to determine resistance to penetration by an indenter at regular depth intervals. The test has a number of modifications that differ according to the shape and size of the indentation device that is used and the force that is applied to them. In comparison to static penetration tests, it enables tests to be made on gravel and other semi-solid rocks. Based on the measured resistance to penetration, the compactness or consistency of soils, the angle of internal friction of cohesionless soils and the unconfined compressive strength of cohesive soils and silts can be determined. At the same time as penetration tests are made, soil samples can be collected for testing in the laboratory. The samples will be used to determine soil moisture content, Atterberg limits, the degree of consistency and the grain size distribution of the soil. The SPT can generally be considered a versatile field test that can be applied to a wide range of geological formations. Some penetration equipment enables logging measurements to be carried out in the penetration cavities, especially by nuclear logging methods.

Large-dimension loading and shear tests are very costly and enable evaluation of the rock mass only at definite points or over small volumes. The validity of extending these measurements to the whole mass is a separate, though not simple question. In addition, it must be borne in mind that these measurements are made mostly in excavations or in underground tunnels, so the results are representative of rocks from areas in which stresses have been released as a result of excavation. The test should therefore be carried out on the least disturbed rock. From this perspective, it is most suitable to carry out tests underground in exploratory tunnels, where the effects of blasting can be countered by breaking down test blocks manually. Another advantage is that tests can be carried out in almost any direction, because the reaction of the loading force can be transferred to the opposite wall or to the back of the tunnel.

An interesting experiment, which was made to assess the effect of the size of test blocks on shear strength, was the shear tests carried out on blocks of large dimensions at the base of the footing of the Mequinenza dam. Geotechnicians are aware that there are a number of factors which influence the results of these tests. They are: the inhomogeneity of the rock mass; variations in the intensity of fracturing; weathering; the state of stress, etc. Normally in Spain, shear tests are carried out on blocks of rock of 0.4 × 0.4 metre or 1.0 × 1.0 metre in size. In order to compare the effect of size on the results obtained, tests were carried out on blocks 4.0 × 4.0 metres in size.

In normal tests on blocks of small dimensions, the full effect of all the characteristics of the rock mass cannot be recognized. The tests described were carried out on Oligocene limestone containing layers of lignite at the Madrid Laboratory of Soil Mechanics. The size of the blocks made it difficult to carry out the tests. The most serious problem was how to achieve the correct distribution of horizontal and vertical forces. It was also difficult to measure deformation of the block caused by the imposed stress. A square block with a four-metre-long bottom edge and a height of one metre was cut from the limestone. At the base of the block, there was a layer of lignite which was tested for shear (Fig. 8.3.6). An L-shaped plate with a height of one metre was concreted onto the side of the block. Vertical pressure was applied through ten flat hydraulic presses with dimensions of 0.95 × 0.25 metre, placed in two vertical rows. Horizontal pressure for inducing shear stress was applied by eight flat presses with dimensions of 1.0 × 0.5 metre. The presses were arranged in such a way so that the resultant force passed through the centre of the lignite layer.

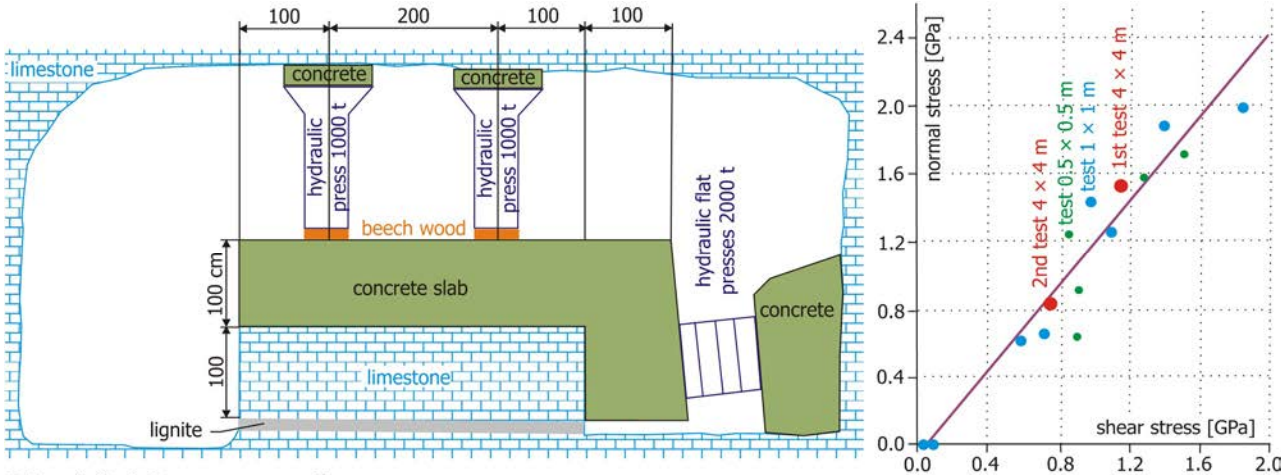


Fig. 8.3.6 Large-area shear test

During the first test, a pressure of 3.6 GPa was achieved. In order to compare the results of the large-scale test with tests on the smaller blocks, shear tests were first carried out on blocks with dimensions of 0.5 × 0.5 metre and 1.0 × 1.0 metre. The tests were made at the same points as those used for the large-scale test. The results are shown in the graph of the relationship between normal and shear stress in the right-hand part of Figure 8.3.6. The relationship between these two variables for tests at all scales is shown as a straight line. This appears not to be correct, because the relationship is not strictly linear. From the results obtained it can be seen that the first test on the 4.0 × 4.0 metre block produced a result that falls above the line, whereas the second test matches the results of tests on the blocks of smaller dimensions. These results lead to important technical and economic conclusions. In soft layered rocks it is advantageous to perform more tests on blocks of small dimensions that yield more statistically reliable values than to carry out tests on large blocks that are several times more expensive and can only be made with great difficulty. It can also be deduced from this large test that the dimensions of the test block should be at least one order higher than the width of the largest discontinuity.

It is difficult to carry out loading tests in the field when long exploratory excavations are being driven by full profile tunnelling machines. Geotechnical testing then comes into conflict with the need to maintain a continuous advance in tunnelling. On the other hand, this tunnelling technique provides better conditions for geophysical surveys of the workings. The results show that, in the case of seismic methods, it is possible to use not only longitudinal but also transverse waves. Figure 8.3.7 shows the results of measurements in such a working. During mechanical tunnelling, the rock mass in the vicinity of a working is not disturbed as much as it is when blasting is used. When applying seismic methods, it is also possible to generate transverse waves using standard seismic sources. As a result it becomes possible to calculate Poisson's ratio and the shear moduli.

In the left-hand part of Figure 8.3.7, there is an illustration of seismic measurements made in a section of the Bystrc tunnel through igneous rocks in Brno. On the right-hand side, a summary of the results is given, including calculations of other parameters for the whole tunnel. If it is possible to "continually" make measurements along an entire working, then the properties of the rock mass can be described statistically by processing the results obtained and thereby objectively divide it into quasi-homogeneous blocks or units. If it is also possible to make other specific geophysical measurements or geotechnical tests, it will be possible to make an even more objective division of the rock mass. A separate subchapter of this book is devoted to the relationship between moduli.

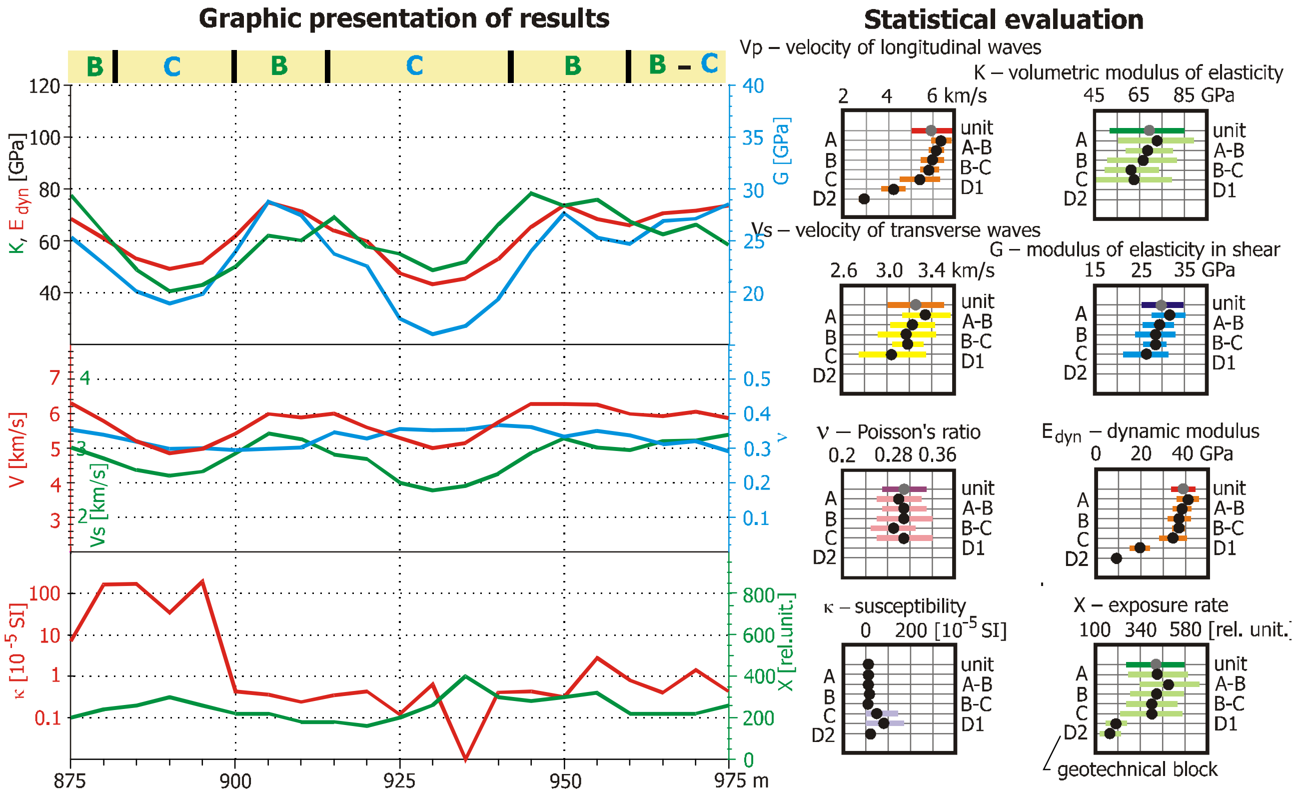


Fig. 8.3.7 Results of geophysical measurement in a mechanically driven working

In order to apply the results of large-dimension tests to the whole rock mass, a great number of mostly rapid and less costly methods have been developed, such as simple penetration tests using cones for indenting the rock, or rebound methods, of which the Schmidt hammer is the best known. A range of laboratory tests are used to determine the physical-mechanical properties of rocks, particularly laboratory shear tests of various types using different configurations for loading of rock samples and triaxial tests carried out on rock cylinders. The determination of certain important physical properties of rocks can also be made directly in the field (the determination of the index of strength by point loading, and the determination of the hardness of rocks using a scleroscope).

Even though a relatively large number of results obtained by these various methods are available and can be correlated with parameters obtained by making point tests, so that the results can be extended to larger volumes of a rock mass, this cannot be considered ideal. Only the developments in geophysical methods and their widespread application have enabled the patterns of zoning of moduli in the rock mass down to the depths necessary for static calculations using the finite element method.

8.3.3 Measurement of the State of Stress in the Rock Mass

To determine the state of stress in a rock mass is a very demanding task, because the natural state of stress is influenced by many factors acting both over the length of geological history and at present. The most important are the field of tectonic stress, the gravitational stress due to the load of overlying rocks, the residual state of stress after denudation by processes of erosion, the effects of the anisotropy of the geological structure and, finally, also the effects of human activity that may have led to additional loading or unloading at a given site.

The main problem encountered when measuring the original state of stress in the rock mass is the fact that there is practically nowhere accessible where the state of stress has not been disturbed to a greater or lesser extent. It is always affected by the disturbance of the natural environment caused when attempting to reach a measuring point. Deep penetration of the rock mass, however careful the technique may be, always causes a disturbance of the original state of stress. Perhaps the only exception is the use of geophysical methods of radiography that do not measure stress directly but the required parameters can be reached by calculation or by using correlation formulae.

In principle, the state of stress of the rock mass can be measured in three ways:

a) The deformation method is based on the measurement of deformation that has taken place after the completion of an exploratory working. The amount of deformation can be measured mechanically using resistance strain gauges or vibrating wire sensors. The value of stress in a given place can be calculated from the amount of displacement of measuring points if the modulus of elasticity and Poisson’s ratio are known.

b) The unloading of drill core is a modification of the deformation method in which the smallest changes in the original state of stress of the rock mass occur. A borehole is drilled in the place where a measurement is required and the dial of a strain gauge is fixed to the ground cross-section of core. After the drilling of another core run (10–15 cm), the readings on the strain gauges are continuously recorded. Poisson’s ratio and the modulus of elasticity are measured in the laboratory using a piece broken from the core. The state of stress in the rock mass is then calculated using all the measurements that have been made.

c) In the compensation method flat presses are used to apply stress to the rock *in situ* (Fig. 8.3.8). The first stage of this procedure is to mount sensors for measuring displacement on the test area and their initial state is recorded. Commonly, several pairs of sensors are used. A groove between

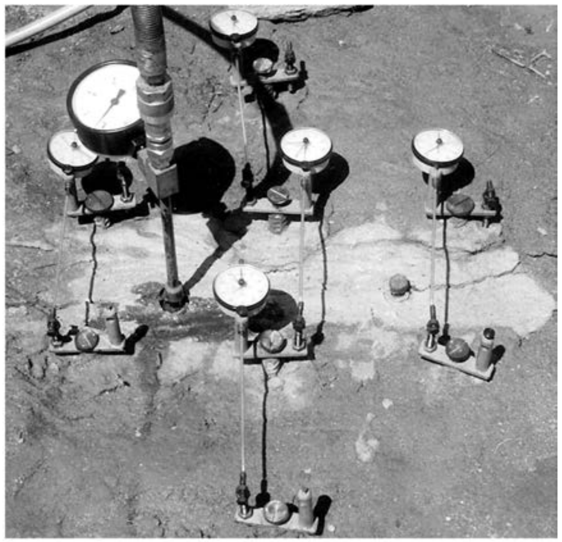


Fig. 8.3.8 Measurement of displacements in flat presses (a photo by O. Horský - 1987)

the sensors is then cut. The distance between the sensors then changes due to changes in the distribution of stress. A flat press is inserted and concreted into position. When the concrete has set, pressure is applied through the flat press and increased until the distances between the sensors return to the original values. This method is suitable only for measuring compressive stresses. Tensile and shear stresses cannot be measured using this procedure. This type of test was used by Geotest to determine the original state of stress in excavations for a power plant and for the Centro Cuba PSHEP project (Fig. 8.3.8).

Certain information about the *in situ* value of stress in the rock mass can also be obtained by using geophysical methods. In this case, however, it is not the value of the stress that is measured, but the changes in its distribution. Special seismic methods, or gamma-gamma logging are used for this purpose. Both methods are often used for monitoring the development of stresses in rock arches in the back of a mining working. An illustration of such a measurement is given in Figure 8.3.9.

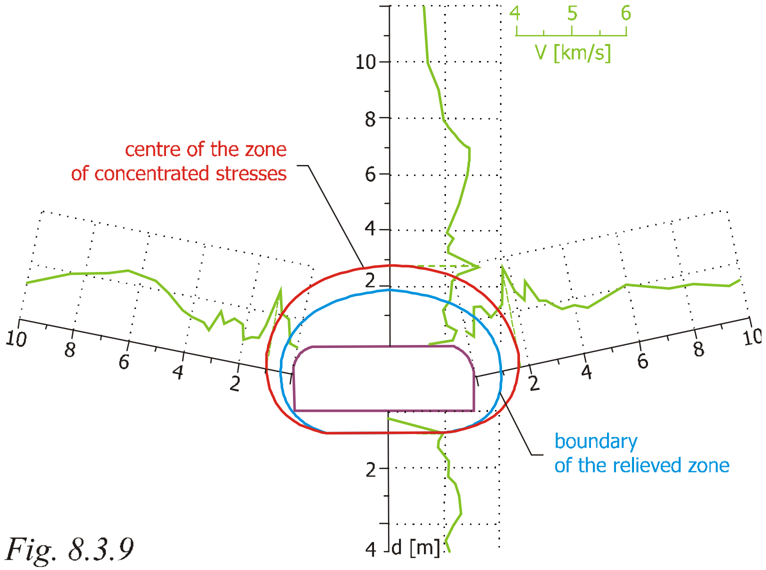


Fig. 8.3.9
Seismic identification of an arch (Votoček et al., 1972)

The distribution of stress in an arch above a test cavity was measured using seismic equipment. The measurement was not made along the axis of a borehole, but by radiography between boreholes spaced 4.5 metres apart; the accuracy of the readout of the beginning of the seismic signal was + 0.01 ms (Votoček, *et al.*, 1972). Measurements on waves parallel to the wall of the cavity were made and the distance of waves ranged from 0.25 metre (in places of marked velocity changes) to 2.0 metres (inside the rock mass). In the original interpretation, the width of the loosened zone around the stope varied from 0.5 to 1.0 metre. Now, with the benefit of greater experience, the estimated width of the loosened mass would be somewhat greater. Even the relatively small spacing between boreholes (4.5 metres) does not prevent the propagation of seismic waves along curved paths. With the exception of the floor, the centre of the zone of concentrated stress is located from 1.0 to 2.8 metres inside the wall of the cavity. The centre of the zone of concentrated stress would not change position even if the paths of the seismic waves were curved. In the case of maximum velocities, the seismic wave does not curve. Similar results would have been obtained using gamma gamma logging, however the use of this method is governed by strict health and safety regulations.

When measuring the state of stress in the rock mass, measurements of the frequency of elastic pulses can also be used. Changes in frequency are caused by changes in the distribution of the original stress or as a consequence of deformation within the rock mass. Elastic waves are generated by energy released during the fracturing of rock. Seismic pulses of very low amplitude are generated when fractures form in a crystal lattice, and this is the case when micro-cracks in the minerals in rocks open and close. The amplitude of these phenomena can be as low as 10^{-10} m, which is the diameter of the hydrogen atom. The frequency of these waves varies over a very wide range from the first tens of hertz up to megahertz. In engineering geology, the use of these waves for geophysical interpretation is called geoacoustics.

Experience has shown that it is possible to use geoacoustics, not only to assess the actual state of stress in the rock, but also to determine the level of stress to which the rock had been exposed in the past by making use of the so-called Kaiser effect. After increasing stress to levels above which the rock had been exposed during its history, the frequency of emissions increases sharply.

Repeated geoacoustic measurements have proved that the development of deformation in a rock stope is incomparably longer than is indicated by convergence measurements. This fact is explained by the sensitivity that can be achieved in geoacoustic measurements. Compared to convergence measurements with an accuracy of 10^{-6} metre, geoacoustic measurements can detect movements with an amplitude up to four orders of magnitude lower. Figure 8.3.10 illustrates the results of repeated geoacoustic measurements in the Viola 2 tunnel in the limestone mass at Hrhov. The lower graph shows how the frequency of acoustic emissions (quantity A_p) decreases with time in all quasi-homogeneous blocks in the vicinity of the exploratory tunnel. In the first period, after the tunnel has been excavated, when the limestone mass begins to fracture, elastic emissions have a higher frequency than in the period when micro-movements along already formed planes of discontinuity prevail. Changes in the frequency agree well with those in static moduli of elasticity measured under laboratory conditions (Pavlík, 1981).

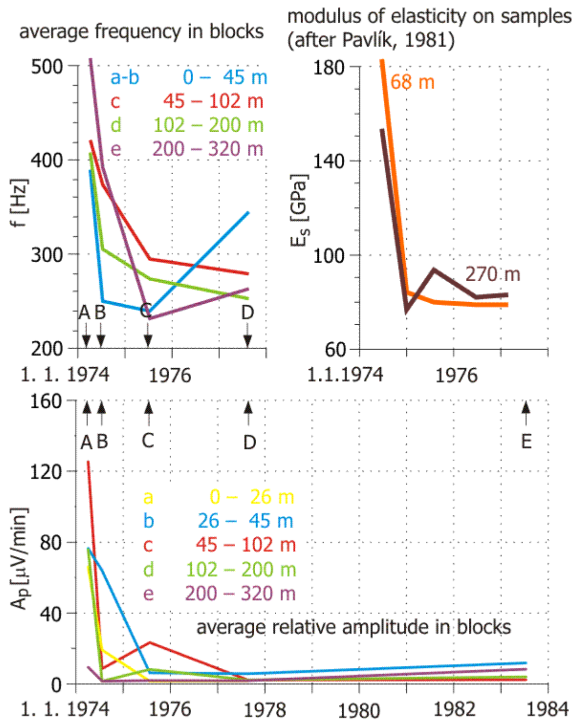


Fig. 8.3.10 Geoacoustic changes and changes in moduli of elasticity

The range of geotechnical tests is very broad and the use of individual types of test changes with time. Equally, it is true that new types of geotechnical tests are still developing and being introduced for survey purposes. This trend is determined by the advances in the technology of measurement and particularly by the use of processors or computers to control the course of tests. New techniques will enable measurement of changes at smaller intervals and will reduce random errors of measurement. Despite these predictions, an overview of the most important tests, and the methods and reasons for their application in surveys for different types of construction and also at different stages of a survey is given in Table 8.3.1 below.

Tab. 8.3.1 Types of geotechnical test

Type of test	A	B	C
Determination of the state of stress in the rock mass and its changes			
• Measurement of convergence of the stope		+	
• Flat presses		+	
• Water pressure blasting tests in boreholes		+	
• Unloading of the face of the stope by borehole or tunnel		+	
• Unloading of drill core (doorstopper, triaxial cell, hollow cell)		+	
• Water pressure blasting test (Hydrofrac)		+	
• Extensometric measurement in boreholes around a stope		+	
• Seismic radiography of the rock mass between boreholes		+	

8.3.4 Tests of Permeability

One of the most important tasks of the engineering-geological survey in dam construction is to ensure impermeability of the dam, not only of the dam body itself, but also of the underlying rocks. It is necessary to seal a permeable rock mass artificially to guarantee the safety of the dam and the economic utilization of the water reservoir. This involves the construction of a grout curtain, especially for concrete dams or high dams of any type. In essence, a grout curtain is an artificial barrier against potential seepage. It is created by injecting cement, clay-cement, clay, or mixtures of chemicals into a fence of boreholes. Simultaneously, by preventing seepage, the amount of upward pressure in the subsoil of the dam is reduced and hence its stability is also increased.

Permeability of the bedrock is commonly measured by means of water pressure test in boreholes. The permeability of the rock mass is governed by such factors as the intensity of fracturing in the rock, the separation of planes of discontinuity, the nature of fracture fillings and other features such as karstic porosity. All these affect the relationship between the loss of water from a borehole and the amount of pressure used. These factors must be assessed systematically. The major factor governing permeability is the ease with which water can pass along fractures and joints. Joints close as depth increases due to the overburden pressure, and particularly in zones of concentrated stress, thus the permeability (Fig. 7.5.2) and pressure conditions in planes of discontinuity also change.

Measurement of deformation characteristics			
• Static loading tests	+	+	
• Tests by deformometric probe (pressiometer)	+	+	
• Tests by radial press (TIWAG)		+	
• Flat presses		+	
Tests of strength of rock mass			
• Shear tests on a predetermined plane of fracturing	+	+	+
• Shear tests on a free plane of fracturing	+	+	
Tests of permeability			
• Water pressure tests in boreholes	+	+	
• Pumping tests	+	+	
• Infiltration tests (Horn)	+	+	
Technological tests			
• Tests of anchorage (deformation and blasting tests)	+	+	+
• Tests of bonding of concrete to rock by shear tests	+	+	
Laboratory tests of rock samples			
• Index tests (bulk density, specific gravity, absorption capacity, moisture)			
• Frost resistance			
• Weathering resistance			
• Unconfined compressive strength (cube strength, cylinder strength)			
• Splitting tensile strength (Brazilian test)			
• Strength under confining pressure			
• Shear strength by a circular die (punch)			
• Unconfined tensile strength			
• Tensile strength under bending			
• Deformation tests			
Explanatory notes:			
A Foundation engineering	*B Underground constructions*	*C Surface excavations*	

Relative permeability of the rock mass is assessed using the results of WPT in relation to a variety of criteria. In this case, the expression of values of the coefficient of permeability has no physical basis because permeability is dominantly controlled by fractures, with the exception of porous rocks that have internal connected porosity. Although there are formulae for the conversion of water consumption in a WPT to a coefficient of permeability (Altovsky, 1962), their application to the rock mass as a whole is not very realistic because the environment is often heterogeneous and affected by discontinuities. Nevertheless, it is useful for calculating the approximate losses of impounded water under the assumption that the rock mass would behave as a permeable porous body.

Lugeon (1933) was one of the first investigators who formulated permissible rock permeabilities in the subsoil on the basis of WPT in boreholes. Thus, today, WPT are commonly described as “Lugeon tests”. The Lugeon criterion is used for dams higher than 30 metres. According to this criterion, the rock environment is considered as impermeable if the specific consumption of water injected into a borehole is lower than 1.0 l/min/m at a pressure of 1.0 MPa. For dams not higher than 30 metres, the Jähde criterion is commonly used; according to this criterion the rock environment is impermeable if the specific consumption of water is lower than 0.5 l/min/m at a pressure of 0.3 MPa. It is necessary to bear in mind that these are conventional values. If another criterion is used for the hydraulic calculation, the value could change. For example, J. Verfel (1974) recommends a certain easing of the Jähde criterion because of the sharp drop of the hydraulic head with increasing depth in the rock mass (Chap. 7.3, Fig. 7.3.13). The procedure for carrying out water pressure and injection tests is fully described in the professional literature.

Pumping and slug tests, measurements of the position of the groundwater level to determine the range of fluctuation, and other hydrogeological procedures used in the survey of dam sites and their reservoir areas have already been discussed in Chapter 5. Other details can be found in the relevant professional literature.

8.3.5 Final Report on the Results of a Geotechnical Survey

The final report on the results of a geotechnical survey must contain the following:

- The purpose and scope of the geotechnical survey, the description of the construction site and the designed structures;
- The procedures used for the geotechnical survey with a special respect to the application of non-destructive methods as a tool for determining correlations;
- A description of the geological composition and structure of the site, giving attention to faults and their effects on the geometry of igneous, sedimentary and metamorphic rocks, and any folds in them and all the related geotechnical data;
- A geotechnical evaluation of the site and assumptions on which the interpretation has been based;
- A summary of the results of field survey and laboratory work, a list of all field and laboratory tests, tabulation of the results and classification with regard to the defined geological structure;
- Groundwater (hydrological) data;

- A summary of the values of geotechnical parameters governing the design of the structure; if correlations are used for determining the values of geotechnical parameters or coefficients, the limits of their validity must be defined; and
- An assessment of the stability of the area, drawing attention to potential difficulties during excavation or blasting work and a plan for remedial or preventive measures.

The final evaluation should also include comments on the geotechnical significance of irregularities in the area of the site affected by construction, such as karst phenomena, pockets or caves, and potential slope failures, drawing attention to the effect they may have on technical procedures used in construction work. Finally, the risk of changes in geofactors of the natural environment must also be evaluated. These may originate during and/or after construction work. A proposal for engineering-geological and geotechnical monitoring of the construction must be submitted.

8.4 Correlations between Parameters

In this subchapter, the relationships between the mechanical and physical properties of rocks (in the sense of geotechnical terminology) will be discussed. The procedures used to make geophysical measurements enable much larger volumes of the rock mass to be investigated than is possible using conventional geological observation and geotechnical tests. This advantage of geophysical methods can be used to acquire a much larger amount of data that can be used for calculations of stability and other geotechnical purposes, and to provide a basis for creating a mathematical model of a studied environment. The value of geophysics is that the validity of geotechnical data can be extended into a much larger volume of rock.

Relationships between mechanical and physical properties can be causal or statistical. An example of the first category is the relationship between moduli of elasticity and the velocities of seismic waves. In the second category, there is no direct causal relationship as, for example, in the case of strength parameters and apparent resistivity. If it is possible to empirically determine sufficiently close relationships between properties that do not have a definite causal link, these relationships can be used to specify the distribution of geotechnical parameters in the studied space but it is important to remember that these relationships will only have local validity.

8.4.1 Causal Relationships

This category includes links between various moduli and the velocities of different types of seismic waves. It might be assumed, for example, that the modulus of elasticity measured by a laboratory test should have the same numerical value as the modulus obtained from seismic measurements. When making a detailed analysis of the procedures used for these two types of tests, it is evident that they are each carried out under different conditions. “Geotechnical” moduli are determined using static loading tests, whereas dynamic moduli of elasticity are obtained by measuring the seismic oscillation of the studied mass. Differences in the numerical values may be as high as tens of per cent in undisturbed rocks; in disturbed, semi-solid rocks and soils the differences in values can be multiple.

Certain factors influence the results of measurement in opposite ways. In seismic field tests, measurements are made on a large volume of the rock mass under investigation, whereas geotechnical tests are made on a small volume of rock that is usually undisturbed. In seismic measurements, the waves pass through both undisturbed rocks and weakened or even disturbed rocks. If the waves propagate perpendicular to such an inhomogeneity or at an oblique angle, the seismic wave will pass through the disturbed environment and "slow down" because of the worse parameters. If a zone of weakness is parallel to the path of the seismic wave or at an oblique angle to it, the seismic wave may even "accelerate" in the zone of concentrated stress around such a fracture.

Another fact that should be taken into account when evaluating relationships is that the setting in which the geotechnical and geophysical tests are made are quite different. In seismic measurements, the wave penetrates an environment with "better" properties, which increases seismically determined moduli. This applies not only to tests on the surface of the bedrock, but also to all the other measurements. The equipment in current use enables the times of arrival of waves to be measured with great accuracy so that it is geophysically possible to study a rock mass of the same order of dimension as that used for geotechnical tests. In numerous cases, measurements of the behaviour of a dam after its construction is completed have shown that moduli determined by seismic procedures are closer to reality than moduli measured by conventional geotechnical techniques.

Another possible cause of differences between moduli of elasticity determined by static tests and by seismic (dynamic) measurement lies in the time over which the tests are made. In seismic measurements, the duration of a test is of the order of fractions of a second and during the test the rock mass experiences not only a compressive load, but also a tensile load and, under the influence of transverse waves, also a shear load. In geotechnical tests, the time is longer by a few orders of magnitude and the tested material is subjected only to a compressive load. Moreover, a compressive test may take hours before the deformation is stabilized, during which creep can occur. This partially counteracts the effect of rock fracturing. Differences of a few per cent have been detected in these types of tests, even in completely compact materials. In the same way as in the case of geoacoustic methods, this effect can be explained in terms of the scale of observation.

There is also a difference in the stresses applied in geotechnical tests and in seismic measurements. In seismic measurements, stresses of the order of 0.1 kPa are used, whereas in geotechnical tests the stress is three orders higher. There are similar differences in the scale of deformation; in seismic measurements it is micrometres, in geotechnical tests on rocks it is millimetres, hence there is a difference of about three orders of magnitude. Theoretical analysis has shown that moduli of elasticity measured in small deformations are higher than those measured in large deformations.

Planes of discontinuity and/or fillings of fractures also have an effect. Ordinary fractures do not have a large effect on the seismic signal, at least as far as velocities are concerned. Fractures might cause attenuation of the signal, but this has no effect on the calculation of moduli. Because the volume of material used for geotechnical tests is smaller by several orders of magnitude, creep on fractures has a significant effect and closure of fractures increases deformation.

Other factors, such as temperature, changes in moisture content, the amount of stress in the rock mass and the surface relief, do not have a strong influence on the relationship between seismic and geotechnical properties. The techniques that can be used to resolve differences between different methods of measurement are described in the literature. In most cases they involve geotechnical tests made over a period of milliseconds (or less) and the introduction of the so-called "initial modulus of deformation". In these cases, the differences between moduli measured by geotechnical and seismic procedures are reduced, however these types of test have not gained favour in broader survey practice.

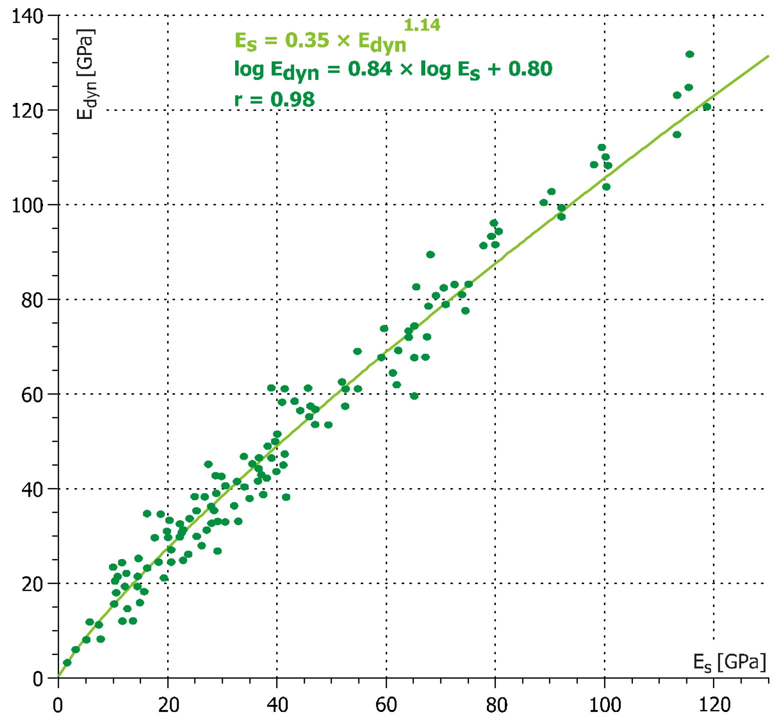


Fig. 8.4.1 Relationship between static and dynamic moduli (after Nikitin, 1981)

Many investigators have described large differences between the two types of measurement. For example, H. Link (1962) stated that seismic moduli in rocks were 3–12 times higher than moduli of deformation measured using conventional geotechnical tests. The equation given by A. I. Savich (1969) is very often quoted and is reproduced in Figure 8.4.1. All investigators agree that there is no universal formula that will enable the values of seismic moduli to be converted into moduli of deformation without limitations. In most cases, consistent relationships have been established for individual types of rocks, but these relationships are applicable only to a given site.

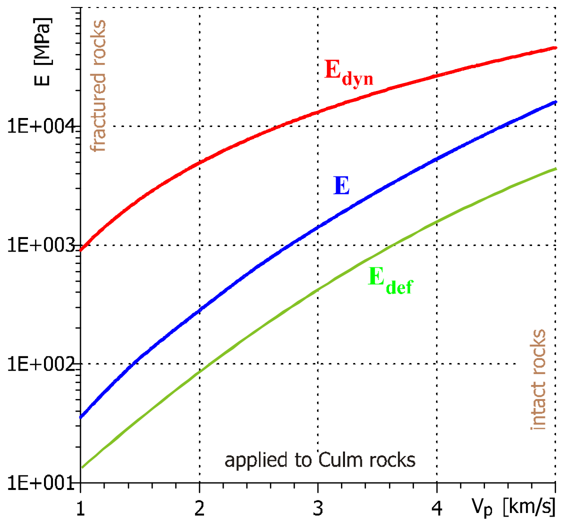


Fig. 8.4.2 Relationship between V and E at Slezská Harta (adapted after Mjuler, Novosad, 1986)

Most of these site-specific studies were made in the former Soviet Union, or in the present Russian Federation. Even recently published books, for example L. Y. Yerofeyev, *et al.* (2006), commonly quote examples taken from the older literature. A graph from the book by V. N. Nikitin (1981) (Fig. 8.4.1) is especially worthy of attention. The relationship illustrated was based on a number of tests made on limestone. Besides graphic relationships, the book also gives a mathematical relationship in which, however, the dependent and independent variables given in the graph are reversed. In contrast to the work by H. Link mentioned above, the difference between the static and dynamic moduli is not so much pronounced. In this case, the dynamic modulus is a little more than ten per cent higher than the static modulus.

In the Czech Republic, and in the former Czechoslovakia, great attention was paid to these relationships. The publications of A. Dvořák (1969) and K. Müller (1987) are among the most notable. A large number of examples can be found in professional articles and papers given at specialist conferences. An illustration of one such approach is given in Figure 8.4.2.

The relationship between the dynamic modulus of elasticity and moduli of deformation are described in terms of the velocities of longitudinal waves in Culm rocks at the site of the Slezská Harta dam. The state of the rocks ranges from disturbed rocks with the character of loose debris to intact rocks. These relationships were determined by comparing geotechnical tests with geophysical measurements made both in the laboratory and on site. Figure 8.4.2 shows that for intact Culm rocks E is lower by about one order of magnitude compared to E_{dyn}, whereas for strongly disturbed rocks it is lower by up to two orders of magnitude. This illustration shows that the conversion of dynamic moduli to static moduli and moduli of deformation is only possible if correlations are made between the same samples or between similar places in the rock mass. Based on the results of seismic and other geophysical measurements, it is possible to make a credible geotechnical division of the rock mass into quasi-homogeneous units for the purpose of other geotechnical tests (Fig. 8.3.7). At the same time, it is possible to assign the necessary parameters to these quasi-homogeneous units at the early design stages before the more advanced stages of geotechnical testing.

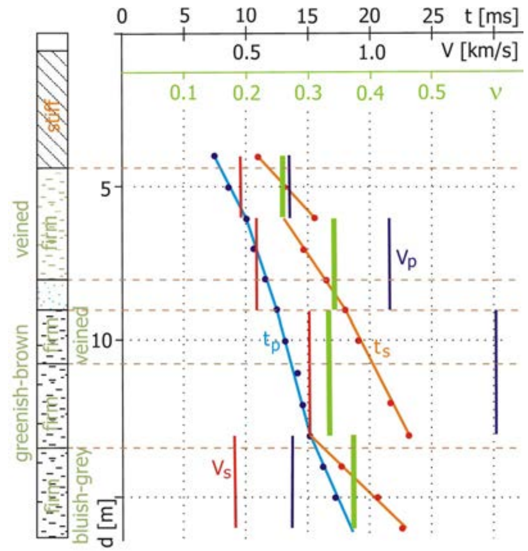


Fig. 8.4.3 Poisson's ratio - seismic measurement

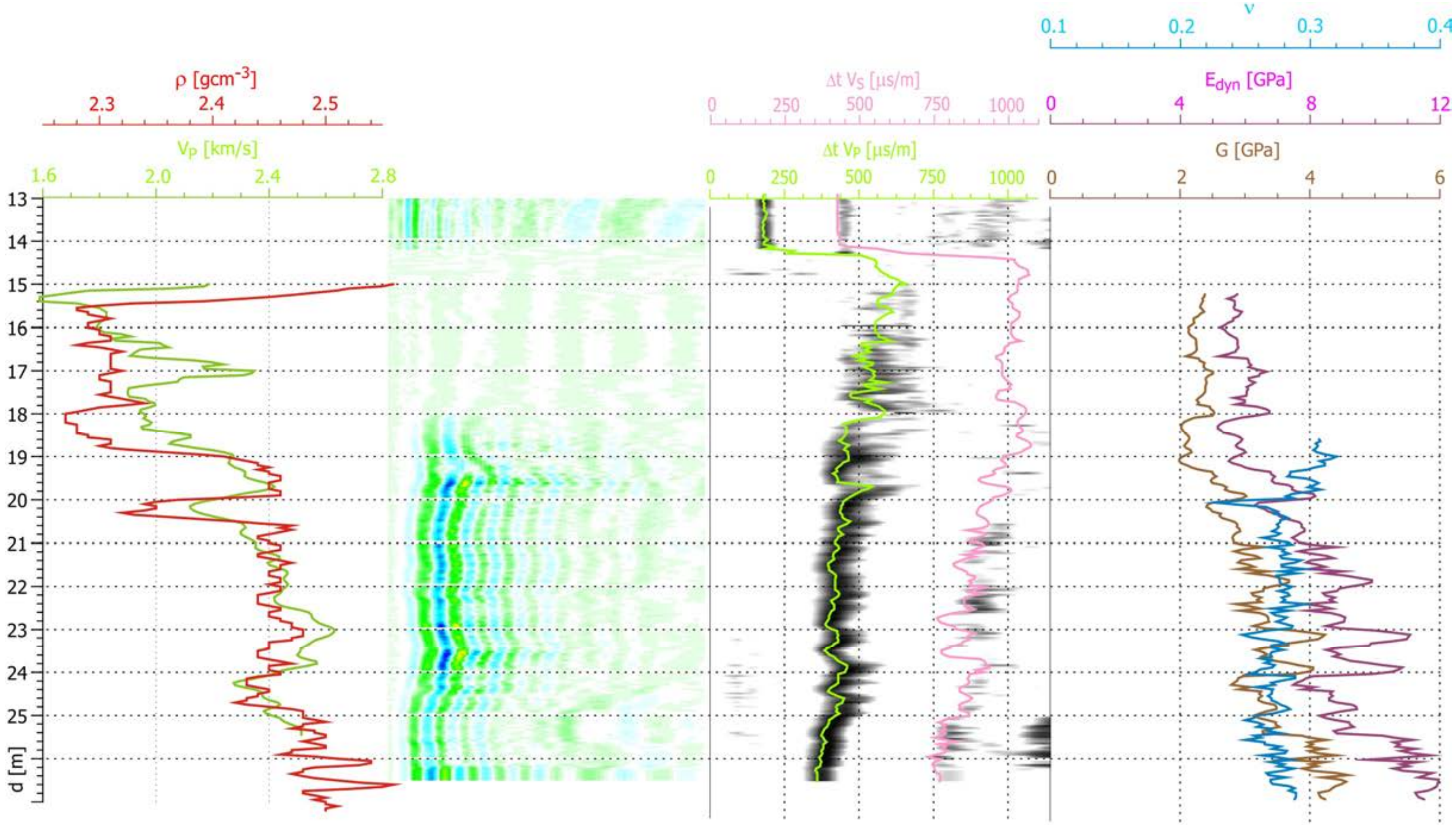


Fig. 8.4.4 Determination of Poisson's ratio in a borehole (adapted after Kořalka, 2008)

Poisson's ratio plays an important role in the seismic determination of moduli. Ideally, this ratio should be measured ***in situ***. In most cases, this requires special procedures using horizontal geophones and a horizontally generated seismic signal. An illustration of such measurements made on

Neogene clay near Vyškov is given in Figure 8.4.3. The pattern of measured values follows the classical shape of the travel-time curve for a longitudinal wave. One surprising observation was the decrease in the velocities of longitudinal waves near the bottom of the borehole. The shape of the travel-time curve for a transverse wave is substantially more complicated. Its shape shows that in the first measurements of the wave it was not possible to construct a continuous travel-time curve, but that in many cases the refraction horizon was the source of a "new" transverse PS wave, which is always faster than the classical SS wave. The measured values of Poisson's ratio at the given site ranged from 0.26 to 0.38, which are normal values for soils of this type.

Logging instruments now available, including modern probes, enable acoustic measurements to be made using not only longitudinal waves, but also transverse waves. This enables geotechnical parameters directly along the axis of a borehole to be determined. An illustration of such measurements is given in Figure 8.4.4 taken from S. Kořalka (2008). These were made using an FWS40 sonic logging probe, manufactured by ALT Luxembourg, to record a full wave image on three sensors at a distance of 0.6, 0.8 and 1.0 m from a transmitter. The measurement was carried out in monotonous clayey shale during a survey of a tunnel for an underground railway line in Prague. A borehole 27.0 m deep was cased down to 14.5 m; the groundwater table was located at 13.0 m.

The measurement of acoustic waves was made all the way down from the casing. Strong attenuation of the acoustic signal was observed down to a depth of 18.5 metres. At shallower depths, the transverse wave was already difficult to detect. In the left-hand part of the figure, the conventional curves for bulk density and the velocity of the longitudinal waves are shown. Based on their shape, it is possible to deduce that the effects of surface weathering of the rock mass reach a depth of 20.5 metres. No signs of fracturing in the rock mass deeper in the borehole are evident. The second column shows a digitized image of the sonic wave. This also reveals that the rock mass is fractured down to a depth of 20 metres. The third column depicts travel times of longitudinal and transverse waves and the black shading indicates the quality of correlation between measurements of the seismic wave on all three channels. The last column shows the changes in the dynamic modulus of elasticity, in the shear modulus and in Poisson's ratio. The value of Poisson's ratio in the intact shale is about 0.35, which is the anticipated theoretical value.

The tomographic processing of seismic radiography enables velocities to be converted to those parameters needed for the design of the dam by using the relationships established. It is always more appropriate to use relationships derived on site rather than relationships taken from the literature, but even the latter can be useful, especially at the early stages of a survey when a general view of the distribution of mechanical properties in the rock mass is required. The illustration in Figure 8.4.5 is from the site at Ipel' where this procedure was used to determine the distributions of the following:

- Index of fracturing;
- Horizontal stress;
- Dynamic moduli of elasticity;
- Static moduli of elasticity; and
- Modulus of deformation.

Figure 8.4.5 shows the distribution of the coefficient of fracturing and the modulus of deformation. The figure also contains the equation and the graph, used to calculate the given parameters. The radiographic measurements illustrated in the figure were made by a team from Stavební geologie Praha. In the case of the modulus of deformation, it is evident that the lowest values coincide with zones of fracturing and, in the case of the coefficient of fracturing, the low values are located in the areas of rock between individual fractures.

As mentioned above, the relationship between the individual parameters can be found in numerous publications, but the relationships are not always clear. In some cases there appear to be shortcomings in the expression of the relationships, and often the details of how the measurements were made are inadequate. Sometimes it is not clear whether the results were obtained from laboratory tests on samples or from field tests, in other cases the lithology of the rocks investigated is not described in sufficient detail and it is not made clear what type of geotechnical tests were used. Typical examples of relationships that suffer from these problems are given in Figure 8.4.6. Although these figures are often reproduced in Russian-language literature, it has not been possible to establish who were the investigators that carried out the original tests or the types of strength tests that were used. It is surprising how large the differences are in the velocity/strength relationship for different types of limestone. At the same velocity, the strength of limestone from an unknown location is seven times

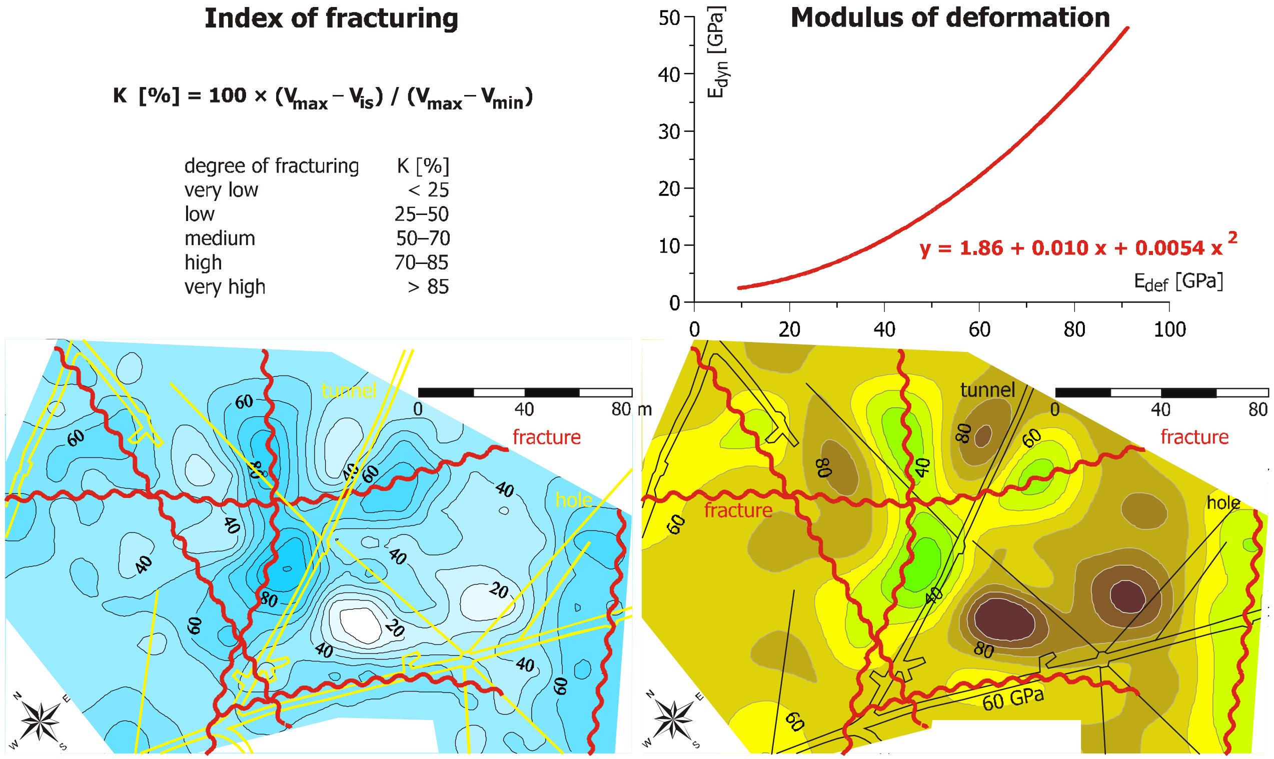


Fig. 8.4.5 Distribution of the coefficient of fracturing and the modulus of deformation at the Ipel' site

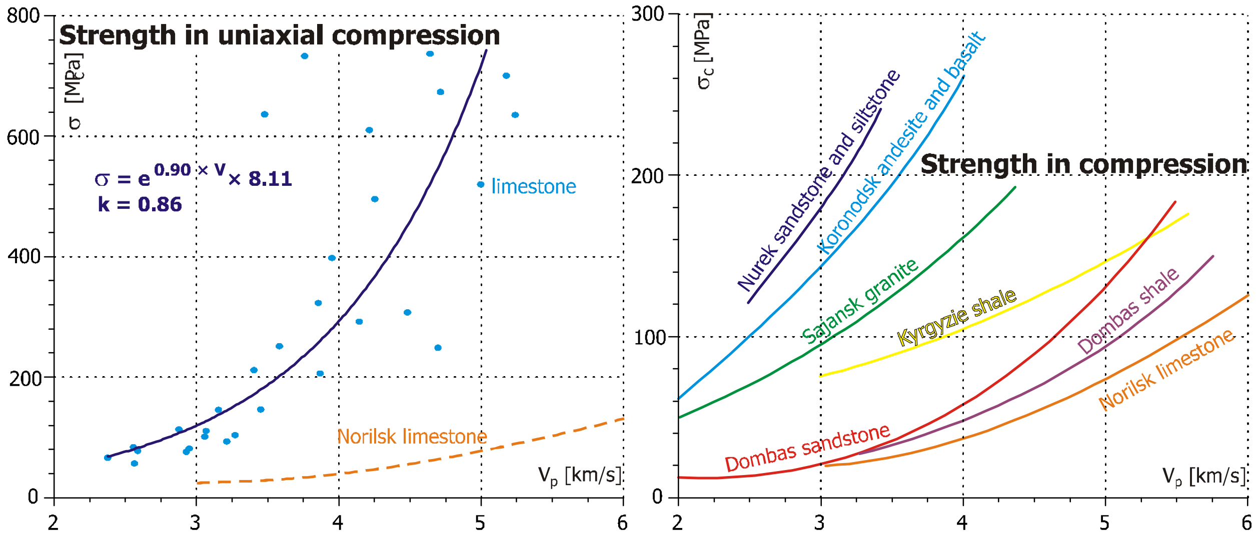


Fig. 8.4.6 Relationship between the velocity and strength (after Zinchenko, Kozak, 2005))

greater than that of “Norilsk” limestone. Under these circumstances, it is impossible to draw other meaningful conclusions about these relationships. It is always necessary to be cautious when taking data from the literature and not to place full reliance on the published relationships. It is always better to establish the correlations that exist at the site of the survey, even though this approach may be more costly and complicated for the engineer contracted to carry out the survey.

8.4.2 Statistical Relationships

In this category, numerous relationships have been described in the literature both in graphic form and in the form of mathematical equations. In some cases, the links between parameters appear to be illogical links (e.g., between dielectric constant, attenuation of seismic waves and borehole yield). For this reason, only relationships that have been used by Geotest for survey purposes, or those that are based on more logical correlations will be discussed below.

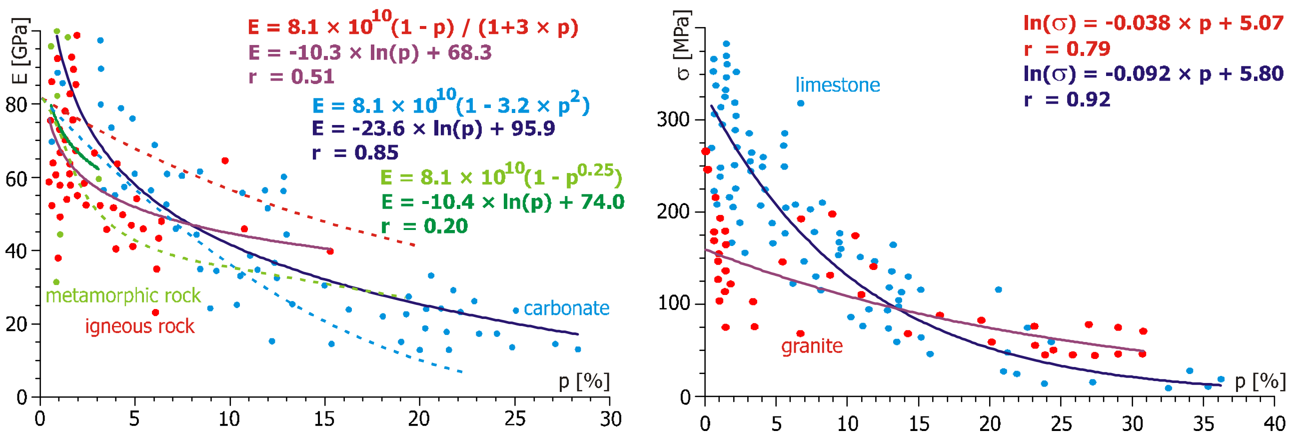


Fig. 8.4.7 Relationship between porosity and the modulus of elasticity and strength (after Zinchenko, Kozak, 2005)

In the first example illustrated in Figure 8.4.7, the relationships between porosity, the modulus of elasticity and strength are shown. In the diagram on the left hand side, the original equations defining the relationships between the parameters are given in red, blue and pale green, together with the relationships in graphic form determined by the authors using the GRAPHER program. Both the diagrams show that there are practically no measured relationships for metamorphic rocks. Only the correlation between porosity and strength for eruptive rocks with porosities higher than five per cent is shown. More detailed relationships are recorded for limestone, or carbonate rocks. Even in this case,

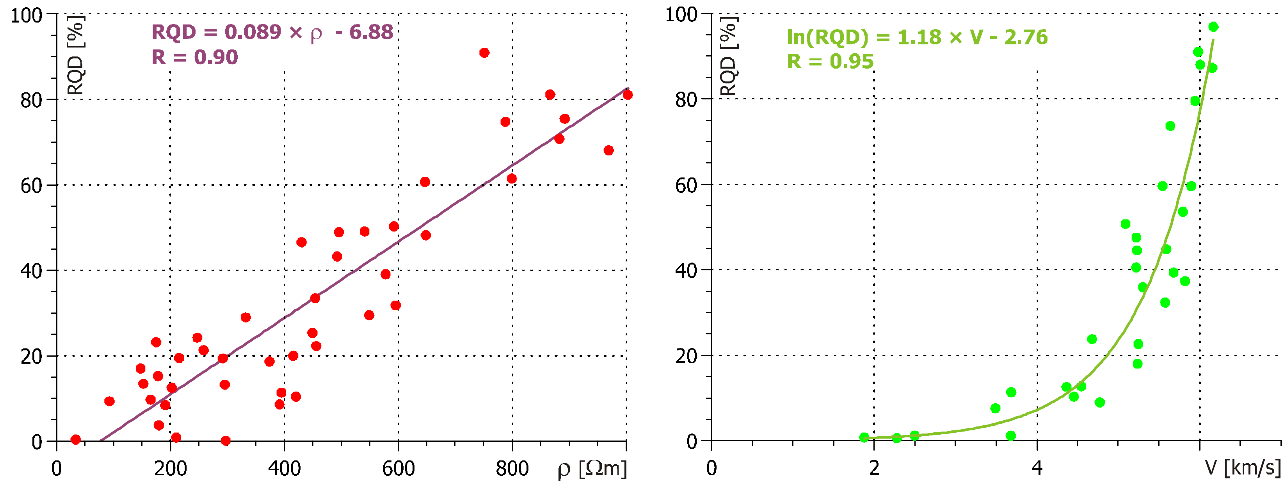


Fig. 8.4.8 Relationship between RQD and resistivity and the velocity of longitudinal waves

however, it is necessary to find a mathematical equation that will define the closest possible relationship. When the population of samples is sufficiently large, it is appropriate to divide the graph into sections and study the relationship in each part.

During the survey for the Dalešice dam, one of the Geotest engineers carried out a detailed study of the various relationships between key parameters, paying special attention to the possibility of determining these parameters using RQD. This became the subject of a candidate dissertation. Figure 8.4.8 shows the relationship between resistivity, the velocity of longitudinal seismic waves and Rock Quality Designation that was established. This shows that a definite relationship exists that can be used for survey purposes.

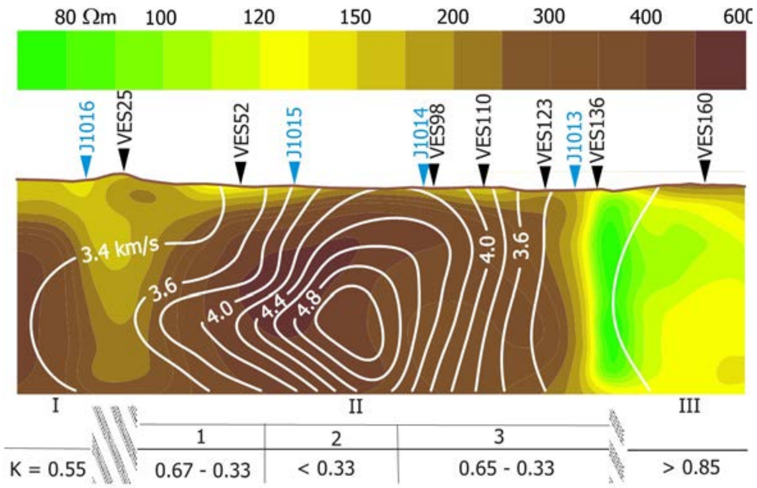


Fig. 8.4.9 Distribution of velocities and apparent resistivity, and the coefficient of fracturing

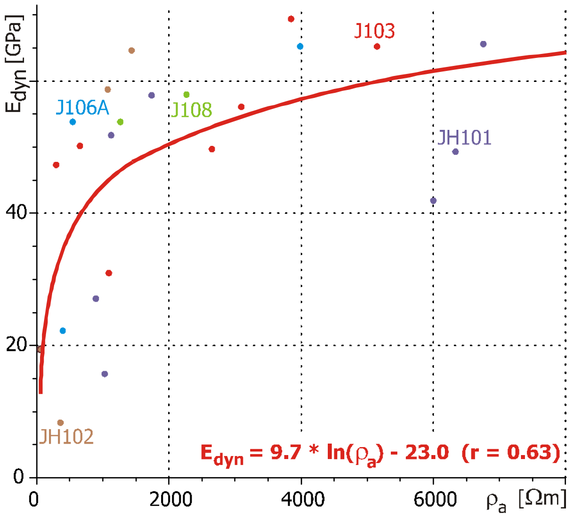


Fig. 8.4.10 Relationship between ρ_a and E_{dyn}

Figure 8.4.9 shows another type of relationship. In this case, the relationship between different parameters is not given in the form of graphs or equations, but by superimposing the patterns of variation on the same section. In this case, the results of seismic radiography between boreholes were compared directly with the results of vertical electrical sounding (VES). The section shows that the patterns of variation in both parameters are similar and that they reflect the same pattern of distribution as the coefficient of fracturing in the rock mass. Both parameters delineate a part of the rock mass (block II) with distinctly better physical properties than the surrounding parts. The definition of areas of weakness in the rock mass, which lie on either side of the block of rocks with better mechanical properties, is clearer in the results of VES than it is in the results of seismic radiography. This is due to the fact that, in the 1970s, tomographic processing of seismic measurements was less advanced than at present. The lowest part of the diagram shows values of the coefficient of fracturing quantified according to the relationship given in Figure 8.4.5.

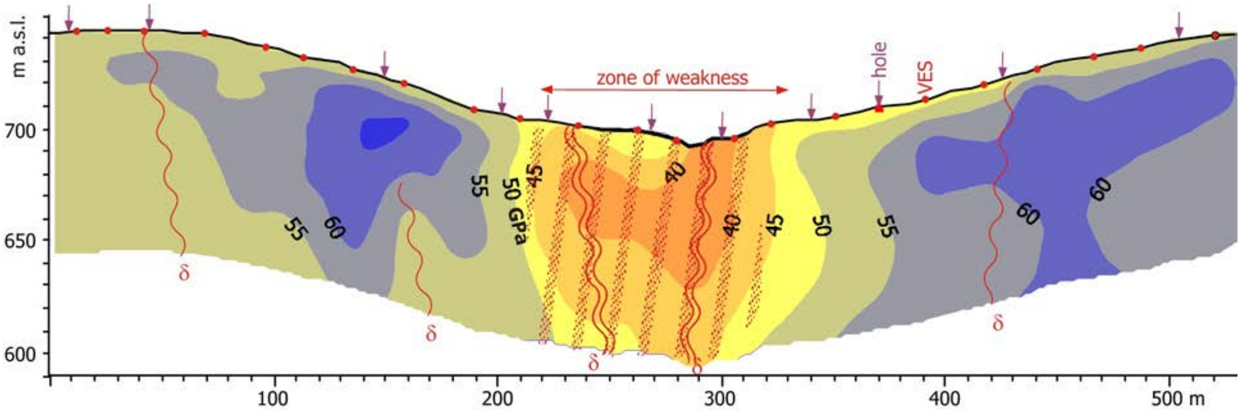


Fig. 8.4.11 Distribution of dynamic moduli of elasticity in the Josefův Důl dam profile

During the orientation survey for the Josefův Důl dam, geoelectrical survey methods were used much more than seismic ones. The reason was the equipment then available to geophysical teams.

In this case, the relationship between apparent resistivity measured using VES and the dynamic modulus of elasticity determined by making ultrasonic measurements on modified drill cores was established (Fig. 8.4.10). The result of this investigation is shown in Figure 8.4.11. Although the correlation coefficient is not high, the relationship was good enough to allow determination of the distribution of the moduli in the dam profile. This procedure enabled the engineering geologists and geophysicists to give to designers a preliminary idea about the moduli of elasticity in the studied granite massif.

8.5 Practical Examples

To solve practical problems in hydraulic engineering, information from many geological disciplines will be used in a geotechnical survey. These include regional, structural and historical geology, stratigraphy, sedimentology, petrography, hydrogeology, geomorphology and sometimes palaeontology, as well as engineering geology. Even though geotechnics is based on theoretical disciplines, such as the science of elasticity and strength, its goal is to obtain practical results that can be used to guide the construction work safely and objectively. Therefore, in this chapter, special attention is given to practical examples of survey work carried out by Geotest. The theoretical background and data relating to geotechnical procedures are the subject of a comprehensive literature, both in Czech and other languages. The same holds true for geophysical methods, which are referred to in this chapter only insofar as they are relevant to the particular examples of surveys described below.

The broad application of geophysical methods to investigate the rock mass has led progressively to the development of a rational combination of geophysical methods that can be used to identify the lithological and tectonic boundaries within it, to assess its physical-mechanical state and to determine the distribution of stress and the processes of deformation that affect it. The description and classification of soils and rocks is always based primarily on the collection of samples and tests made on them in the laboratory. By combining comprehensive laboratory tests with geophysical information, the geotechnical properties of the rock mass required for the design of the project can be compiled at the preliminary stage of the survey. At the detailed stage of the survey, greater emphasis is placed on field tests and on establishing the correlations needed to determine derived characteristics and recommended design values. The scope of these tests and the measurements made depend on the character of the designed structure in relation to the rock environment.

Despite the possibilities offered by new instrumentation and the increasing speed with which the results of field and laboratory tests can be acquired, the evaluation of the studied rock mass and the overall success of a geotechnical survey finally depend on the combined experience of the whole team of engineers and technicians involved. The geotechnical description of the rock mass remains the foundation for the successful completion of any dam project, but it is not the only factor involved. Nowadays, projects of almost any scale and technical difficulty are conceivable, provided that there are funds to carry out the work. However, the more thorough the survey of the site of a future dam is, the fewer the technical uncertainties will be. This decreases the chances of miscalculations and accidents that can be the cause of drastic increases in the cost of a project as well as in the time required for its completion. In this chapter it is the intention to show how necessary it is to make use of all available methods to gather the maximum possible information about a site and how the effective evaluation of this information depends on full cooperation between all the specialists involved. The maxim adopted by Geotest is that the most valuable lessons are learned by finding practical solutions to real problems. For this reason, a number of geotechnical and engineering-geological investigations made by Geotest at different sites will be described below.

The first example chosen is that of the Dalešice dam site, where a survey was originally carried out for the construction of an arch dam (1968–1971). Later, because the geotechnical conditions for constructing an arch dam were judged unsuitable, a new survey for an earth-fill embankment dam was carried out (1971–1972).

The dam site is underlain by basic metamorphic rocks, chiefly amphibolite with conformable bands of biotite amphibolite and biotite migmatite and lenticular bodies of ultrabasic rock. The band of amphibolite on which the dam was founded is about 500 metres wide (Fig. 3.3.1). A canyon-shaped valley about 120 metres deep with steep sides cuts across the metamorphic sequence. The amphibolite form pronounced ridges, while weaker rocks in the sequence form distinct depressions covered by slope sediments. The range of geological and tectonic processes imposed on this sequence of rocks with different mineralogical compositions and physical properties, together with the wide range of observations that were made, meant that a geotechnical model had to be built in order to solve the geological-tectonic problems at the site (Fig. 3.3.5).

The model at a scale of 1 : 500 consisted of three horizontal perspex plates equally spaced. The whole model was attached to a wooden base-board. The positions of exploratory workings, especially boreholes, pits and tunnels and the contours of the topography of the valley were marked on the perspex plates. The information about exploratory workings marked in colour was limited strictly to objective facts. In the marked exploratory workings, the lithologies of different types of rocks were distinguished by colour, and different degrees of metamorphism were indicated by fine shading. Zones of mechanical (tectonic) disturbance in individual types of rocks were also marked and, in the case of boreholes, core recovery and intervals with distinct water losses were recorded. The geotechnical model created by marking all this information in its true three-dimensional relationship enabled various scenarios for the interpretation of the data to be tested, for example the position of faults and the contacts between different types of rocks.

The choice of scenarios for the geological structure was progressively reduced by the addition of new information from other exploratory workings and by structural measurements in tunnels and pits. These were continuously assessed by comparison with the topography and with the results of the geophysical survey. The working model of the dam site enabled the division of the rock mass into quasi-homogeneous blocks consisting of different lithologies and bounded by persistent planes of mechanical discontinuity. Since the time when the survey at Dalešice was carried out, major advances have taken place in computer-assisted drafting procedures enabling all the recorded relationships to be displayed and manipulated in virtual space. The building of models has now been replaced by virtual modelling in three-dimensional space and the compilation of data in geographic information systems. Despite the ease with which data can now be handled, careful compilation and interpretation is still necessary.

A detailed statistical evaluation of the tectonic features in each of the identified blocks was made, using stereographic plots and rose diagrams (see Fig. 2.3.17). It should be borne in mind that if one outcrop or test block is chosen for field tests, then the structure can be depicted graphically using simple stereograms. If, however, it is necessary to determine the structural characteristics of the whole mass of a quasi-homogeneous block, a statistical analysis of all structural measurements is required. For this purpose, a strict genetic classification of the measured structural planes and lineations is required. It is important to establish both the morphological characteristics and the relative ages of different generations of joints, foliations, folds and faults as well as those of bedding planes, etc. This is a very demanding task, because the measurement and description of structural features and their graphic depiction is only the first step towards interpreting the structural evolution of the geological environment and its likely geomechanical behaviour. The spatial distribution of structural data may not be uniform, so it is not easy to resolve the patterns clearly, even when a large number of measurements have been made (Fig. 3.3.4). In fact, the task of structural analysis was only completed successfully by combining geophysical measurements with the information gathered by direct observation of the structural features in surface and underground exposures and in drill core. The specific objectives of the structural survey were as follows:

- To identify zones of weakness in the amphibolite mass at the dam site and in the granulite mass in the area of a powerhouse (mechanically weakened zones, conformable interbands of biotite amphibolite, biotite migmatite, isolated lenticular bodies of ultrabasic rock, faults filled with cataclasite and clay gouge, etc.); and
- To observe and record geological relationships and structures in exploratory workings, tunnels and boreholes and to make measurements so that physical parameters, particularly moduli of elasticity, could be specified.

At the survey stage for investment in the dam project and for comprehensive design of the project, an orientation geophysical survey of the dam site was carried out. This survey was made using a limited number of profiles and the results could be used only for a schematic correlation of the most important faults. At the survey stage for the comprehensive design of the project, detailed surface geophysical measurements were made. These were complemented by measurements in tunnels. Based on the analysis of all measurements, the physical properties of the main types of rocks were determined (Tab. 8.5.1).

Table 8.5.1: Physical properties of rocks as a basis for selection of surface methods

Rock	g_0	κ	r_z	I_γ	V
	[gcm^{-3}]	**[10^{-3} SI]**	**[kWm]**	**[mr/hr]**	**[kms^{-1}]**
Compact amphibolite		÷1	2–5	6	3.6–4.5
Biotite amphibolite	2.95–3.15	< 5	1–5	6	
Tectonically fractured amphibolite		< 45	1–1.5	12–20	1.5–2.7
Ultrabasic rocks		15–55	0.1–0.4	2–6	0.9–1.2
Migmatite		1	0.1–0.25	16–30	0.6–1.0
Granulite	2.65–2.70	1	0.05–1.45	7–14	2.0–5.2
Infilling of tectonic fractures		1	0.1–0.15		0.3–1.0

The delineation and interpretation of zones of mechanical weakness was made mainly on the basis of the results of resistivity profiling, or by taking into consideration the anomalies of the vertical component of the magnetic field in the places where ultrabasic rocks occurred. Observations made in exploratory workings (boreholes and pits) were used in conjunction with the results of the geophysical survey to trace individual zones of weakness and faults and to plot their positions in three dimensions on the working model. Mainly strike fractures were correlated and identified by numerals from 0 to XII. This correlation and interpretation was verified by a 270-metre-long cutting at the foot of the slope on the right bank of the valley (Fig. 3.3.7). Fold structures, especially the outcrops of the most obvious ultrabasic bodies, were traced by joining the maxima of magnetic anomalies. Steep fractures, labelled A-G, formed during the youngest stage of faulting in the amphibolite mass contain infillings of clay and clayey breccia. These fractures were indicated by decreased values of apparent resistivity and it was possible to interpret their positions reliably using both resistivity profiling and resistivity sounding.

In those places where the spacing of boreholes was not sufficiently close, i.e. at the edge of the amphibolite mass close to the contact with the granulite, the positions of the persistent fractures D and E were interpreted using only the results of the geophysical survey. At the same time, the contact zone between amphibolite and granulite was geophysically defined and later verified by the cutting on the right bank, the tunnel Št-14 and the diversion tunnels OŠ-1 and OŠ-2.

For the project design, in addition to laboratory and field tests of moduli of deformation, it was necessary to obtain an idea of the moduli of elasticity throughout the whole of the amphibolite and granulite masses, in particular at the base of the dam footing and other facilities. For this purpose, the measurements of velocities of seismic waves and thus also of dynamic moduli of elasticity were made as follows:

- Seismic sounding and profiling along the floors of exploratory workings;
- Seismic measurements on outcrops;
- Seismic radiography between boreholes, or between boreholes and tunnels; and
- Ultrasonic logging in certain exploratory boreholes.

Modified seismic sounding and profiling measurements were made in tunnels using a single-channel Terra-Scout seismic apparatus with an impact source. In seismic sounding, phases of individual types of waves were observed and travel-time curves were constructed. Boundary velocities were calculated from the travel-time curves of waves refracted at the boundary between the loosened zone and the unaffected zone and, in some favourable cases, the values of Poisson’s ratio, ranging between 0.23 and 0.26, were determined from the behaviour of transverse waves. The dynamic modulus of elasticity was then derived from the values of the boundary velocity and the bulk density. Seismic profiling in tunnels was carried out by measuring the apparent velocity of waves over a constant distance between the source and the sensor. This enabled the zones of minimum velocities to be defined and thus also individual zones of mechanical weakness in profiles along the tunnels and beneath the floor of tunnels. The results of seismic measurement in tunnels were correlated with the results of the measurement of apparent resistivity along the floors and walls of tunnels and with the results of the measurement of magnetic susceptibility and the dose intensity of gamma radiation along the walls.

Surface seismic measurements were also made along a cutting on the right bank (Fig. 3.3.7). After being geologically described, resistivity profiling was carried out along this cutting so that the section could be divided into physically quasi-homogeneous blocks and detailed seismic measurements were made in the individual blocks using the minimum distance between the source and the sensor of waves. Using the values of velocities of longitudinal waves, dynamic moduli of elasticity were calculated for each of the physical blocks (Müller, *et al.*, 1970).

To obtain an idea about the dynamic moduli of elasticity, seismic radiography was carried out on the left as well as on the right bank between boreholes on the one hand and between boreholes and certain tunnels on the other (Pertoldová, 1968). This measurement enabled the anisotropy of velocities of waves in different parts of the amphibolite mass to be determined so that the depth of weathering of the rocks could be estimated. Seismic radiography (Müller, *et al.*, 1971) was only used in the granulite mass in the area of the excavation for the powerhouse, however this gave a much better indication of the distribution of velocities of seismic waves in the mass and the moduli of elasticity. The measurement was carried out using a sixty-channel seismic apparatus designed for oil exploration. Radiography was carried out between fourteen boreholes along eight planes at depth intervals of five metres. This methodology enabled both the horizontal and vertical changes of velocities and moduli of elasticity to be determined so that the extent of mechanical weakness in the mass could be assessed. The spatial distribution of the results of seismic radiography were correlated with the results of resistivity sounding, because the decrease in the velocity of waves and the decrease in apparent resistivity are both a function of the degree of mechanical fracturing of the mass and so these two methods can be combined to produce complementary pictures of the mechanical state of the rock.

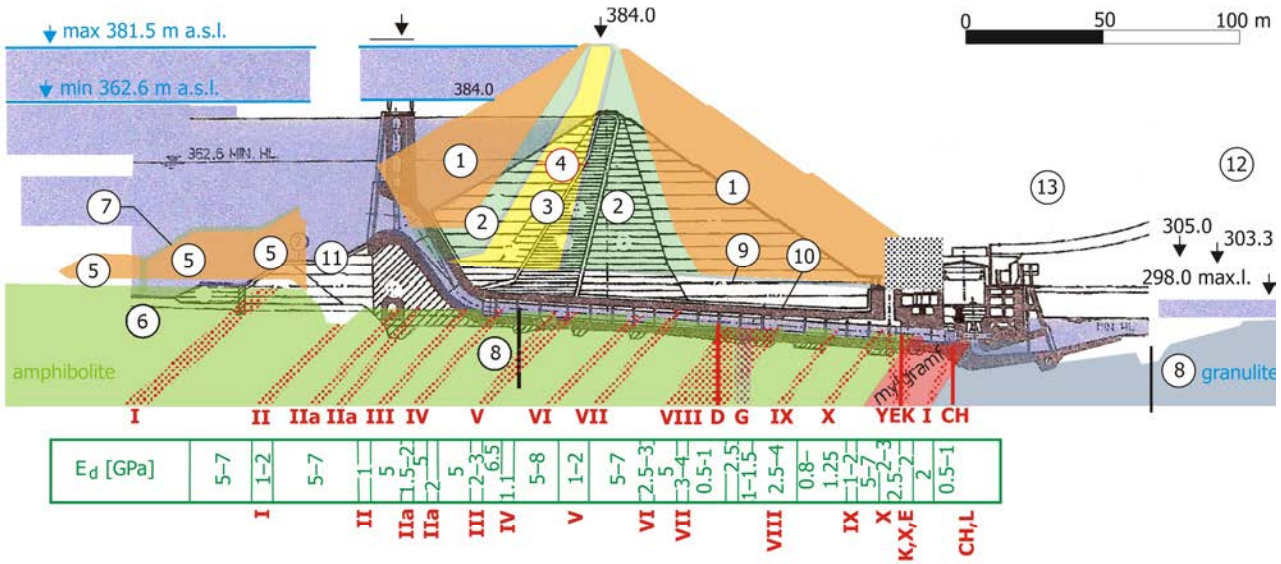


Fig. 8.5.1 Longitudinal section of the intake, penstock and pumped storage hydroelectric plant of the Dalešice dam; 1 – rock-fill part of the dam, 2 – gravel transition part, 3 – loamy core, 4 – sand filters, 5 – upstream pools, 6 – watertight concrete diaphragm, 7 – watertight concrete facing of the upstream pool, 8 – grout curtain, 9 – reinforced-concrete constructions of penstocks, 10 – steel penstocks of 6,200 mm diameter, 11 – intake, 12 – pumped storage hydroelectric plant 13 – power lines from the PSHEP

Certain boreholes were measured by ultrasonic logging (Pantl, 1969). These measurements along the axis of boreholes enable mechanically weak zones to be identified and to be correlated with intervals of poor core recovery. The velocities of seismic waves and the dynamic moduli can be calculated. Samples were taken from boreholes for laboratory measurement of the velocities of waves using ultrasonic radiography to determine E_{dyn}, static E, and moduli of deformation E_{def} using the method of Šrejner. These tests are necessary to establish the relationship between dynamic moduli and static and deformation moduli determined using different procedures. To distinguish between different lithologies and permeable units in a borehole, ultrasonic logging was complemented by resistivity logging, by the method of infusion and dilution and by nuclear logging (GL, GGL – Valtr, Pantl, 1971).

The combination of methods used for the geophysical survey enabled physically quasi-homogeneous units limited by individual faults of particular generations in the amphibolite mass and in the granulite mass to be defined. In addition, the most probable values of physical parameters for these units, particularly dynamic moduli of elasticity, were calculated. Based on the identified correlations and relationships between dynamic moduli of elasticity determined by different methods in the laboratory and ***in situ***, it was possible to provide the designer with reliable information about changes of moduli of elasticity and other physical parameters in the vertical and horizontal directions in the rock mass. A summary of these parameters is given in Figure 8.5.1.

The states of stress in the rock mass were the most difficult of the physical-mechanical characteristics to measure. The first data were acquired by analysing the results of measurements made in boreholes, particularly sonic logging measurements, and the results of seismic radiography made between boreholes (Fig. 8.5.2). Figure 7.5.2 showed the results of sonic logging from a borehole in the granulite mass that indicate the presence of a zone of concentrated stresses at a depth of 30 to 40 metres beneath the valley of the River Jihlava. This zone is characterized by maximum values of the velocity of longitudinal waves. Also the patterns of specific losses of water and reduced core recovery according to RQD in relation to depth also indicate zones of concentrated stresses. The results of ultrasonic logging also provide an idea about the distribution of stress in the rock mass, but only in the close vicinity of measured boreholes. The spatial distribution of stresses was only revealed by the results of seismic radiography between boreholes. This gave a complete picture of the distribution of stress zones beneath the bottom of the valley.

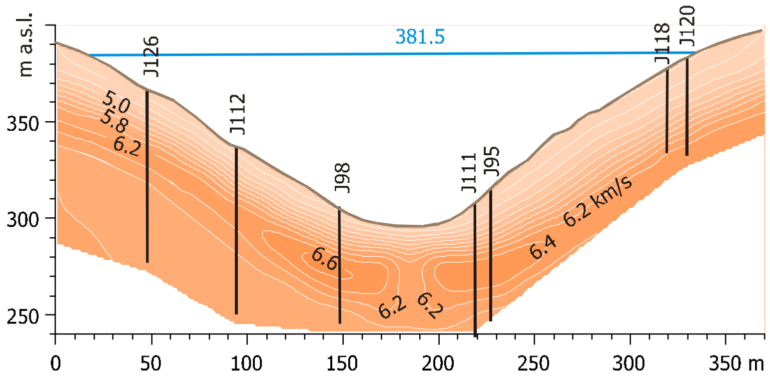


Fig. 8.5.2 Velocity maxima below the Dalešice valley bottom

In this way, three belts of stress were identified in the granulite mass:

- A belt of decreased stress manifested by lower velocities, the thickness being about 20 to 30 metres, in which there was an increase in the separation of planes of mechanical discontinuity caused by the loosening of blocks of rock and by the formation of non-tectonic joints and fractures due to the release of stress and gravitational effects;
- A belt of increased stress characterized by maximum velocities of longitudinal waves, the thickness being about 10 to 15 metres, in which the rocks show a reduction in the frequency of mechanical fracturing; and
- A belt of normal stresses, which must be considered as variable over the long-term evolution of the valley, both in terms of the extent of mechanical fracturing and the magnitude of the stress.

Seismic measurements in the granulite mass (Figs. 8.5.2 and 8.5.3) show that there is a zone of maximum velocities at the foot of both of the slopes at a depth of about thirty metres; this was interpreted as a zone of concentrated stress. It is clear that this interpretation provides only a general indication of the state of stress in the studied mass, but an attempt was made to determine the state of stress of the studied mass more precisely, using a procedure that had rarely been used before in geotechnical surveys for large engineering constructions.

This new method, used for the first time on a Czech project, required expensive field tests to be used to determine the geotechnical properties of the rock mass. The tests carried out were as follows:

- In diamond boreholes: pressiometric tests over a range of stress up to 2.5 MPa using a Menard pressiometer (Ø 60 mm) down to a depth of 20 m below the base of the footing of the arch; and
- In tunnels and pits: classical loading tests on an area of 0.2 to 0.5 m^2, in different orientations relative to the foliation and relative to the course of the base of the footing:
  - o Radial loading tests (applied using the method of the company TIWAG);
  - o Large-area expansion loading tests with a loading area of 1.5–1.8 m^2 and measurements of the pattern of deformation and stress in the rock mass (Fig. 8.5.4);
  - o Shear tests on rock blocks;
  - o Corner shear tests; and
  - o Shear tests on planes of mechanical discontinuity.

The aim of the loading tests was to determine the deformation properties of the rock mass in the base of the footing of the dam and other facilities (objects) and to determine their variation with increasing depth (down to about 50 m) in different orientations relative to the planes of foliation. The broader application of results to the rock mass was based on seismic measurements using the relationship between the velocities of waves and depth below surface to compile contours of seismic velocities of longitudinal waves in a profile drawn through the dam axis. Deformation moduli were derived from these.

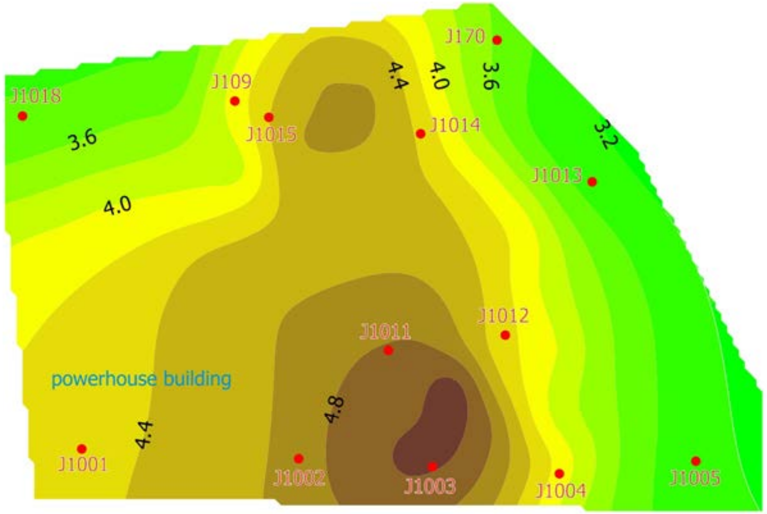


Fig. 8.5.3 Velocity contours thirty metres below the Dalešice valley

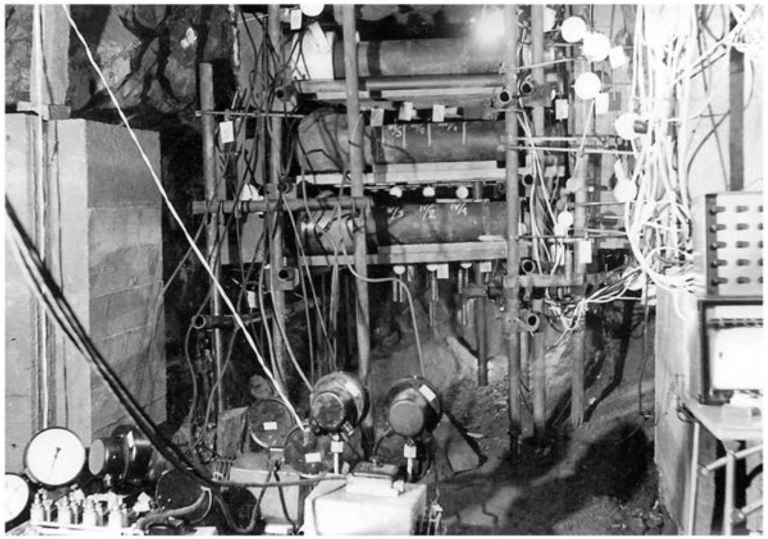


Fig. 8.5.4 Large-area two-directional pressure test (a photo by O. Horský - 1986)

Shear tests were used to determine the strength of the rock as a whole and also the strength of the rock along planes of mechanical discontinuity in the rock mass so that the strength on the main planes bounding the individual quasi-homogeneous rock blocks and inside the blocks could be measured. Three main types of tests were used:

- Tests made on the corners of a loose block of rock, the size of the loaded block usually being 1.0 × 0.7 metre and the maximum imposed pressure 600 MPa; in this type of test, the rock is free to fail by shear along a plane, so shear or rupture occurs along the weakest direction in the loaded rock (Fig. 8.5.5);
- Shear tests carried out in frames to measure shear strengths on foliation planes, joints and fractures; steel frames with dimensions of 50 × 50 cm or 20 × 10 cm were used; and
- Tests made on loose blocks of rock, usually of dimensions 80 × 50 cm to 80 × 100 cm and about 50 cm in height, resting on the floor of a tunnel. The blocks are loaded with a normal force N and pushed by a force T from the side so that the resultant passes through the imaginary centre of the shear plane; this type of tests was used where the rock was intact or where jointing was not well-developed.

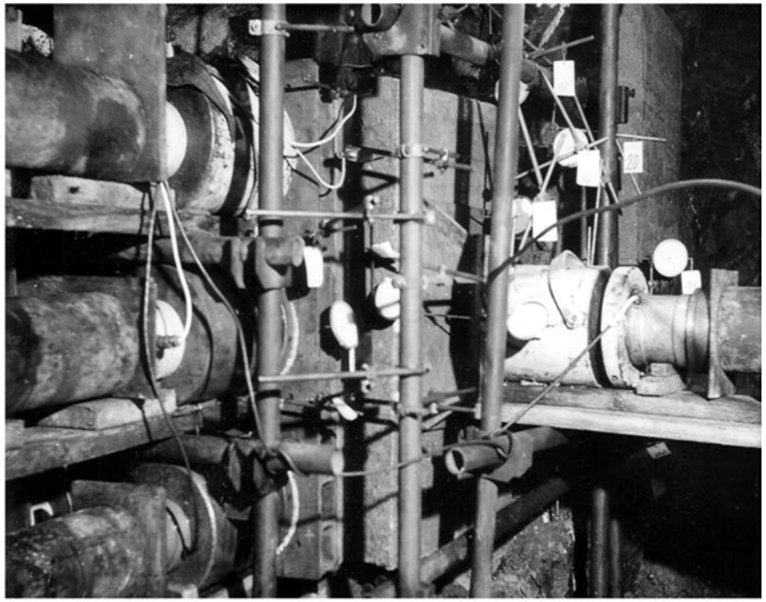


Fig. 8.5.5 Corner shear test (a photo by O. Horský - 1986)

For each type of rock, the cohesive strength and the angle of internal friction at the yield point were determined by interpreting the results of the shear tests. In particular, design values at which shear would occur partly along a joint or along the strike of planes of foliation were established for each type of rock and for the rock mass. Also, the shear strengths of the infill of strike faults, and of jointed and tectonically deformed rocks were also calculated.

An overview of the scope of the main types of survey work is given in Table 8.5.2, in which a comparison is made with a similar survey for one of the world’s highest arch dams on the River Inguri. The extent of the additional survey work required for the final earth-fill embankment dam option chosen for the dam is also given.

Based on the relatively large number of tests of deformation and strength characteristics carried out *in situ*, it was possible to determine zones in the distribution of these values in the area of the base of the footing of the arch dam. The graphs showing the increase of moduli of deformation with depth below the ground surface and comparison with the values of pressiometric moduli enabled an estimate to be made of the deformation properties in the area beneath the footing of the dam.

The wider application of the results to the whole rock mass, especially in the area of the embankment earth-fill dam and the powerhouse, would not have been possible without the application of indirect, mostly non-destructive methods. Although the strengths and moduli of core samples from some boreholes were determined in the laboratory using Šrejner’s method, these values were measured under different conditions than those determined *in situ*. Therefore, geophysics, in particular seismic methods, were used extensively so that correlations between the unconfined compression strength of rocks, the modulus of deformation and the dynamic modulus of elasticity could be determined; correlations between rock quality designation and the dynamic modulus of elasticity; and between the resistivity of the rocks and the modulus of deformation were also investigated. Even though the inter-relationships between these values cannot be generalized, they are demonstrable in the given rock mass and enabled the zoning of geotechnical properties both in the area of the anticipated base of the footing of the dam and at different levels below the ground surface in all the types of rocks occurring, including zones of tectonic disturbance. The estimated values of the moduli for different types of rocks and for different degrees of fracturing are given in Tab. 8.5.3.

Table 8.5.2: Comparison of survey work at dams

Type of working or test	**Dalešice 115 m arch**		**Inguri 270 m arch**		**Dalešice 110 m rock-fill**	
	[pc]	**[m, m^3]**	**[pc]**	**[m, m^3]**	**[pc]**	**[m, m^3]**
Exploratory boreholes	125	5941		8110	24	940
Tunnels	13	738		4500	2	132
Pits and trenches	35	429		680	10	70
Stripping	1	3500		4560	1	270
WPT – number of storeys	480		500			
Classical loading tests	74				6	
TIWAG loading tests	4		40			
Large-area, expansion loading tests	12					
Tests by Menard pressiometer	42					
Shear tests of rock blocks	5				4	
Shear tests	6		9			
Shear tests on planes of discontinuity	9					

In addition to these tests, a Schmidt hammer was also used to measure the rock strength, and indentation tests using Šrejner's method were made. Moreover, relatively comprehensive laboratory tests of the physical-mechanical properties of rocks were also carried out.

If the combination of survey work used at the Dalešice dam site is critically evaluated in terms of the extent to which the requirements of the designer were met, the conclusions are as follows:

- The quasi-homogeneous units of the rock mass were completely delineated using the main tectonic surfaces and petrographic boundaries;
- For each of these units, the required geotechnical parameters were determined and the mechanical discontinuities, deformation properties and strength characteristics were described but the extrapolation of measurements of deformation properties obtained by making point field tests to greater depths within the mass was not completely satisfactory. Also, it was not possible to determine the changes in these properties in relationship to the direction and value of the stresses acting on the rock;
- The permeability of the rocks and their injectibility under the prevailing natural conditions was determined; neither appropriate equipment nor a verified procedure for assessing the permeability of the rock mass under the load imposed by the dam was available at the time of the survey; and

- In the 1970s, at the time when the survey was carried out, it was not possible to fully characterize the state of stress in the rock mass under natural conditions; however, a procedure for determining the field of stress in the characteristically homogeneous mass was developed by using changes in velocities of elastic waves.

Figure 8.5.6 is an aerial view of the body of the Dalešice dam and the pumped storage hydroelectric plant. There are neither signs of weakness in the rock mass nor of any of the lithological contacts at the site, and there are no signs of slope failures.

Geophysical methods can be used to advantage for geotechnical description of a designated dam site, not only at the more advanced stages of a survey when special methods are used, but also during the orientation survey. This approach was used successfully when making a survey for dam sites on the River Genal. Of course, geological field trips were made to determine various physical properties at rock outcrops and the cores from previously drilled boreholes were also used. This provided some information about the behaviour of rocks and their properties. Table 8.5.4 gives the moduli of elasticity acquired by making parametric ultrasonic measurements on drill cores from the area of the proposed dam profiles on the central part of the course of the River Genal, supplemented by moduli calculated using the mean velocities measured in the individual types of rock.

At first sight, it is clear from the values given in Table 8.5.4 that fracturing, when present, has the strongest effect on velocities. The velocities of longitudinal waves passing through a crack are reduced by 35 % (i.e. 65 % of the original value). Foliation also has a strong effect. The coefficient of anisotropy reaches up to 2.0 and can be expected to be higher in reality. This is because the measurements do not cover the whole ellipsoid of anisotropy but

Table 8.5.3: Comparison of moduli

Rock	**1**	**2**	**3**
	[GPa]	**[GPa]**	**[GPa]**
Amphibolite (sound intact)	18–40	33–52	30
Amphibolite (sound to moderately jointed)	8–18	6–20	4–20
Amphibolite (intensely jointed)	2–5	2–5	0,6–5
Fault rocks	0.3–1	1.2–2.2	0.6–3
Migmatite	1.5–2	1–2.2	1.5–2.5
Ultrabasic rocks	0.5–6	3–2.5	
Granulite (moderately jointed)	10–12	30–50	
Granulite (intensely jointed)	8.5–9.5	5–30	

Explanatory notes:

1	*Modulus of deformation E_{def} on the strike or generally oblique to foliation at a depth of 30 m below ground*
2	*Seismic modulus E_{dyn}*
3	*Pressiometric modulus E_1*

Fig. 8.5.6 Aerial view of the Dalešice dam

were made in only two directions determined by the orientation of the borehole axis in relation to foliation. Based on the results of parametric ultrasonic measurements, it was possible to infer the state of fracturing and weathering of the rock from the results of shallow seismic refraction measurements, though only limited information about the lithological composition of the rocks could be deduced.

Figure 8.5.7 illustrates the distribution of resistivities and velocities measured in a survey of four dam profiles on the River Genal. Both the distributions clearly show that three main types of geological environment are present. By comparing these results with observations made on geological field trips, it was possible to establish that the three environments were disturbed rocks (regardless of lithology), phyllite and schist. The same division into three groups was obtained by statistical evaluation.

Table 8.5.4 Measurements of moduli of elasticity made on drill cores and in situ

Hole	h	Lithological type						Remark
		Phyllite		Schist		Quartzitic schist		
	[m]	Along hole	Along foliation	Along hole	Along foliation	Along hole	Along foliation	
Measurement in samples:								
C1	52			32	43			Across fracture 9.1
C1	68					40	89	Across fracture 33
C2	15			18	19			
C2	21	42	55					
C2	33			15	11*			Intensely fractured
C3	73			59	48			Across fracture 36
C3	75			54	50			
C3	85			39	67			
C4	48					54	57	
C4	59					60	58	
C6	82	3.7	46					Across fracture 3.0
C6	99	4.5	49					
Average		12	50	31	45	51	60	Across fracture 17
Anisotropy			2.0		1.2		1.1	
Measurement *in situ*:								
			Phyllite	**Schist**		**Fractured rocks**		
			30	44		16		
Explanatory notes:			Moduli in the table are given in GPa					
			* A large amount of graphite present in this sample					

An important task of engineering-geological and geotechnical surveys is to provide data in a concise form that can be clearly understood by both the clients and the technical specialists responsible for taking further decisions about the fate of a project. This principle was followed in the presentation of information about possible profiles for a dam in the valley of the River Genal. The evaluation of each of the proposed profiles was compiled in a single table (Tab. 8.5.5), which was submitted to the competent Spanish authorities. On the basis of this information, profile P1 was chosen for the next survey stage.

The geoacoustic method yields valuable information about the distribution of stress in the rock mass. As noted above, it is possible to obtain some indication of zones in which compressive or shear stresses are concentrated (Figs. 8.3.9 and 7.3.4). Another illustration is given in Figure 8.5.8.

The red curve in Figure 8.5.8 shows the mean value of the relative amplitude in a block of intact gneiss around a giant opencast brown coal mine. By the middle of 1987, the values of the relative amplitude corresponding to the background noise in the rock had been recorded. In the autumn of 1987, as the exploitation in the giant opencast gradually approached the edge of the high slopes of the mountains, the first geoacoustic anomalies started to appear. These anomalies were not stable either in space or in time. This circumstance is also evident in the mean values of the whole gneiss block. The size of the anomalies gradually increased up to the end of 1989 when a value of 17.1 μV/min was reached. At about the same time, the first anomalies also appeared in stable, highly sensitive inclinometers. Therefore, a geoacoustic measurement was carried out in a tunnel near them. This showed the same level of the geoacoustic field (the blue column in Fig. 8.5.8). By comparing the results of both methods it was discovered that the geoacoustic measurements indicate changes in the distribution of stress in the rock mass up to three years earlier than the most sensitive direct geotechnical measurements.

One of the most demanding contracts undertaken by Geotest was the engineering-geological survey for the Centro Cuba pumped storage hydroelectric plant (see Chapter 4.4, Figs. 4.4.5 and 4.4.6), where, due to a pronounced inhomogeneity of the basement, the determination of the moduli of deformation in the foundations of the dam for the upper reservoir was a very difficult task requiring an especially complicated procedure.

Table 8.5.5 Concise summary of essential information about dam profiles on the River Genal

1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**Remarks**
P1	L		1	B	4	8	18	35	1.	
	U	1		F		8	15	50		
	P		1	B+F		10	22 (to 30)	40		
P2	L	1	1	F+B	7	22	45	65	3.	Indications of creep on left bank
	U	1		F		5	15	50		
	P		1	B+F		13	20 (to 30)	40		
P3	L			B+F	5	15	25	40	2.	Intersection of two systems of faults in valley
	U	1		B+F		8	10	60		
	P	2		B+F		15	25	40		
P4	L	1	2	B+F	8	10	20	35	4.	Intersection of two systems of faults in valley
	U	1		F		5	13	60		
	P	3	2	B+F		13	30 (to 60)	50 (to 80)		

Explanatory notes:

1	*Dam profile*				
2	*Side of profile*				
3	*Number of prominent faults and fractures*				
4	*Less significant faults and fractures*				
5	*Lithology (B = schist, F = phyllite)*				
6	*Total number of faults in dam profile*				
7	*Thickness of Quaternary sediments*				
8	*Depth from surface to relatively sound bedrock*				
9	*Depth of permeability based on the Lugeon criterion*				
10	*Order of preference for dam profile based on selected criteria*				
L	*Left side*	*U*	*Valley*	*P*	*Right side*

The rocks that underlie the dam profile to a depth of 15 to 20 metres are metaterrigeneous schist of the Chispa Formation. To a depth of 4–6 metres, the schist has been weathered to red eluvial silty clay; lower down to a depth of 10–20 metres, it passes into partly weathered and tectonically deformed semi-solid rocks with their original structure preserved. Lithologically, it is muscovite-quartz schist, locally graphitic, which forms a coherent remnant in the mantle of deeply weathered rock. Even though the schist appears to be solid rocks when newly exposed in excavations, it quickly degrades into soils if it is broken and moved. Beneath it, there is a "transitional" layer, about five metres thick, in which beds of semi-solid rocks of the Chispa Formation alternate with more solid calcareous beds in the underlying Cobrito Formation, which is a sequence of metacarbonates (marble, marble breccia, crystalline limestone and calcareous schist with a variable content of graphite, transitional to graphitic schist with intercalations of chlorite and silicate schist). These rocks are strongly folded and tectonically disturbed.

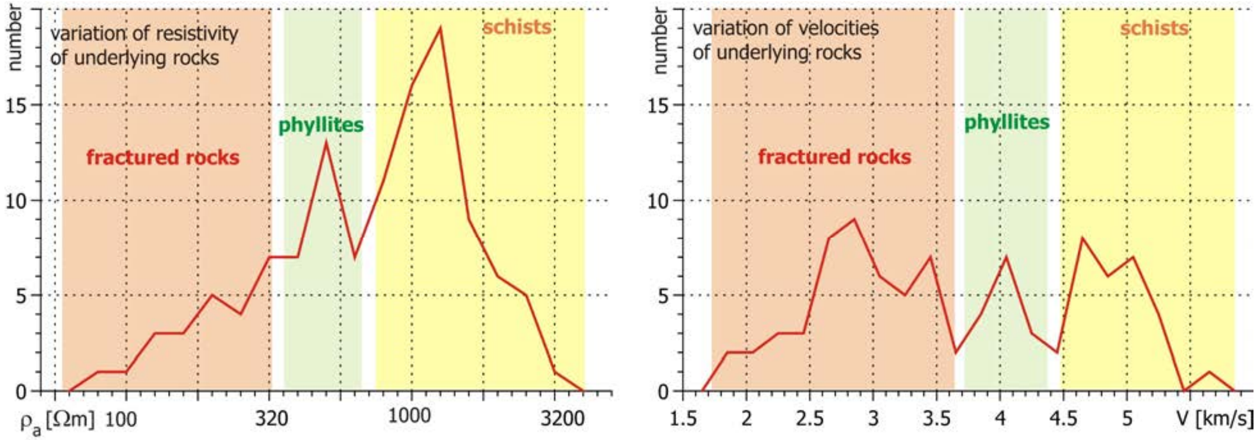


Fig. 8.5.7 Variations in resistivity and velocities of longitudinal waves respectively

A series of faults pass through the dam site (Fig. 3.3.3). They have engineering-geological significance because they are planes and zones of weakness that affect the construction of the dam. Because tectonic movements along these faults occurred repeatedly, zones of crushing and intricate folding of more plastic beds are associated with these faults. During deformation, graphite migrated into these zones of increased stress, thus a high degree of graphitization can be observed on fractures. The faults, though mechanically weak, do not show high levels of permeability because they were compacted and silicified due to the remobilization of chemical components during hydrothermal events that accompanied tectonic movements.

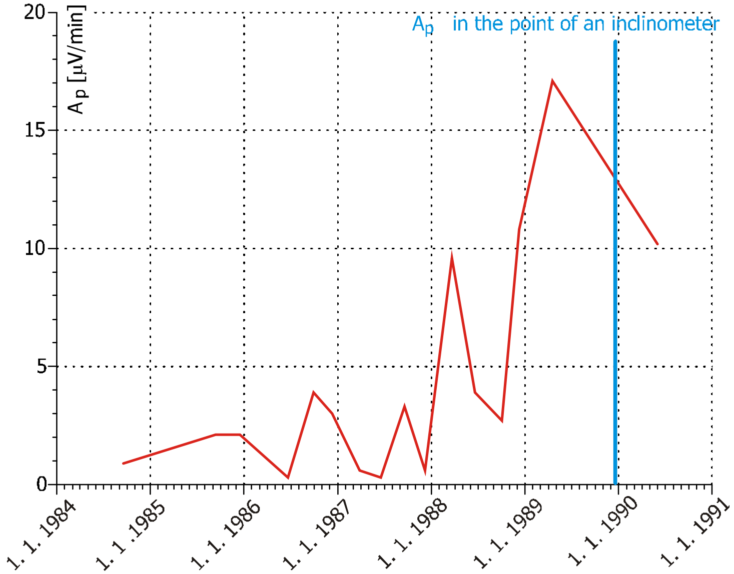


Fig. 8.5.8 Concentration of stress, and geoacoustic activity

Due to the geological complexity and structural history of the area and the tropical climate, the rocks at the dam site were intensely deformed and deeply weathered. In order to define a sequence of quasi-homogeneous units along the profile through the dam subsoil in such complicated geological conditions, it was necessary to use a combination of field survey methods (loading tests ***in situ***, pressiometric and penetration tests), laboratory tests (on monolithic samples collected from the Chispa eluvium and from drill cores) and indirect near-surface geophysical survey procedures in the 'borehole – surface mode' as well as in borehole logging.

To obtain a "velocity" model of the rock environment, seismic radiography was carried out between pairs of boreholes in the dam profile using a procedure in which the area between boreholes was divided into segments in which the velocities of the seismic wave were calculated from wave velocities by the method of weighted averages (Vlastník, 1987). The individual segments were assigned a Poisson's ratio based on the type of rock and the values of the dynamic moduli of elasticity were subsequently calculated. In addition, the results of seismic radiography were used to construct graphs showing the relationship between depth and velocity and between depth and the change in dynamic moduli. These relationships were recorded separately for each borehole (Fig. 8.5.9). It turned out that the minimum increase of velocities and moduli was characteristic of tectonically disturbed areas and of calcareous-graphitic schist, the intermediate values for graphitic-calcareous schist, and the maximum values for crystalline limestone and marble. Similar relationships were also shown by the mean values for groups of boreholes. It was also obvious from the shape of the curves that the reduction in the increase of moduli occurred, on average, at a depth of 40 metres, which corresponds to the depth of partial weathering of the rock; below this the rock is intact and further increase in the moduli takes place.

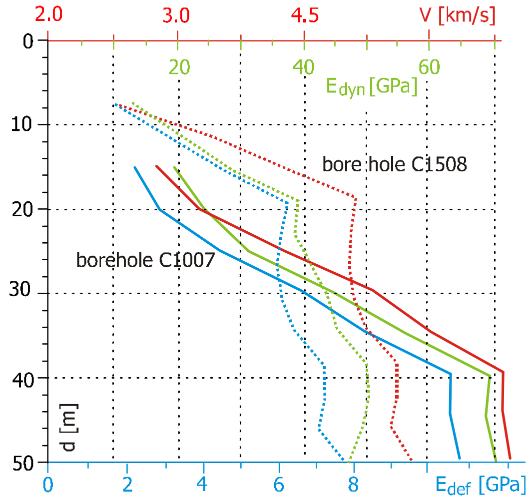


Fig. 8.5.9 Velocities and moduli growth

Based on the results obtained, contours of the moduli were constructed. There was complete agreement with the geological structure, the degree of weathering and the degree of fracturing of the rock. There was also good agreement between seismic radiography and surface seismics, where the first refraction horizon mostly corresponded with the surface of the heavily weathered bedrock, the second one with the depth of weakly weathered rock, and the third one with the depth of slightly weathered rock or the surface of sound rock. The relationships between velocity, the dynamic moduli of elasticity and the deformation moduli calculated for different depths in the rock mass corresponded well with the results of field loading tests.

Table 8.5.6: Relationship between E_{dyn}/E_{def}

Rock	Index		Modulus of deformation		Dynamic Modulus of elasticity	E_{dyn}/E_{def} ratio	
	Geol.	Geoph.	Perpendicular [GPa]	Length-wise [GPa]	[GPa]	Perpendicular	Length-wise
Graphitic-calcareous schist	5	2–2a	6.2	12.5	50	1:8	1:4
Calcareous-graphitic schist	6	2b	0.7	8.3	40	1:57	1:5
Average values of moduli	5	2–2a	9.35		50	1:6	
	6	2b	4.5		40	1:9	

Loading tests *in situ* were made in an exploratory tunnel at depths 50 to 60 metres below the ground surface,

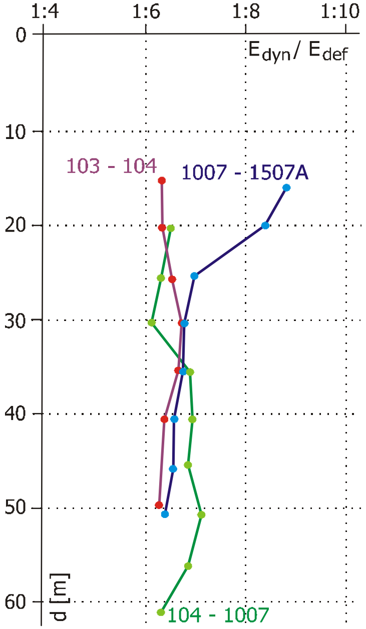


Fig. 8.5.10 Relationships of modulis

i.e. in sound rock. For sound rock, the average values of the modulus of deformation (regardless of the orientation of foliation) and the dynamic moduli of elasticity are in the ratio of 1:6 and in the ratio of 1:9 for heavily weathered (or intensely graphitic) rocks. The values of the moduli of deformation obtained from these tests are in good agreement with the values of the moduli derived by calculation from seismic velocities. In this case, similar ratios between the moduli, 1:6, and 1:9, were obtained (Fig. 8.5.10, Tab. 8.5.6). It should be noted once again that the correlations obtained only have local validity specific to this particular site. However, they can also be used as an approximate guide to the behaviour of rocks at other sites with similar geology.

Based on the application of the correlations between parameters established in this way, it was possible to make realistic predictions about the variations in the moduli of deformation at the dam site down to a depth of 60 to 70 metres and recommended design values of these moduli for rocks in different states of weathering were submitted to the designer (Tab. 8.5.7).

Table 8.5.7: Recommended values of moduli

Degree of weathering	**Velocity V**		**Dynamic modulus**		**Modulus of deformation**		**Average depth**
	1	**2**	**1**	**2**	**1**	**2**	
	[km/s]	**[km/s]**	**[GPa]**	**[GPa]**	**[GPa]**	**[GPa]**	**[m]**
Heavily weathered	3.0	2.4	25	7	1.4	0.8	< 20
Slightly weathered	3.8	3.0	30	18	2.0	1.2	25
Partially weathered	4.2	3.4	40	28	3.0	2.0	40
Intact rock	> 4.7	> 4.4	> 40	> 36	> 4.6	> 3.8	> 40
Explanatory notes:	*1*	*Graphitic-calcareous schist*					
	2	*Calcareous-graphitic schist*					

Table 8.5.8: Comparison between moduli of elasticity assessed by WPT and by seismic radiography

Bore-hole	**1**	**2**
	m	**m**
101	30	30
102	35	35
103	27	25
104	71	40
106	32	40
115	13.5	
1007	40	40
1501	42	40
Explanatory notes:		
1	*Permeability of rock: WPT*	
2	*Weathering of rock: seismic radiography*	

The results of the comprehensive assessment of the rock mass and particularly of the values of the velocities of longitudinal seismic waves confirm the agreement between the depth of the permeable zone and the depth of rock weathering or fracturing. The results obtained indicate that other, less demanding, less costly and notably faster methods might be used to assess the depth of permeability of the rock mass.

This clearly shows that only a comprehensive and careful evaluation of all the survey methods used and the determination of correlations between laboratory and field tests can lead to the correct evaluation of results (Tab. 8.5.8). It is emphasized once again that the correlations established between parameters are always site-specific.

The engineering-geological survey of the Centro Cuba PSHEP was a very demanding task, because the whole area of interest had been affected by a large number of geological processes during its geological history, the most recent of which obliterated the evidence of the preceding processes to a significant extent (Quaternary neotectonic movements, tropical weathering, and deep erosion of sediments). A thorough understanding of the geological development of the region and the specific phenomena affecting the area of interest turned out to be of fundamental importance for the evaluation of point field tests and geophysical measurements and for their application to the whole rock mass, paying particular attention to the effects of stress. The generalization of observations and the application of the results to the definition of quasi-homogeneous units would not have been possible in this case without close cooperation between engineering geologists, geotechnicians, geophysicists and regional geologists.

A major task of the survey was to identify a suitable site for a powerhouse. The original proposal was to locate the powerhouse underground in a hall excavated in calcareous-graphitic schist. Because of their tectonic history and composition, these rocks proved to be intensely fractured, unstable and susceptible to caving. When an exploratory tunnel was being driven, several cave-ins occurred. Deep boreholes collared higher on the slope to test the structure intersected a continuous bed of calcareous schist and marble, so it was decided that the hall for the powerhouse could be excavated in these rocks (Fig. 8.5.11). When the exploratory tunnel was lengthened, intensive faulting and karstification of the mass was discovered so this option was also rejected.

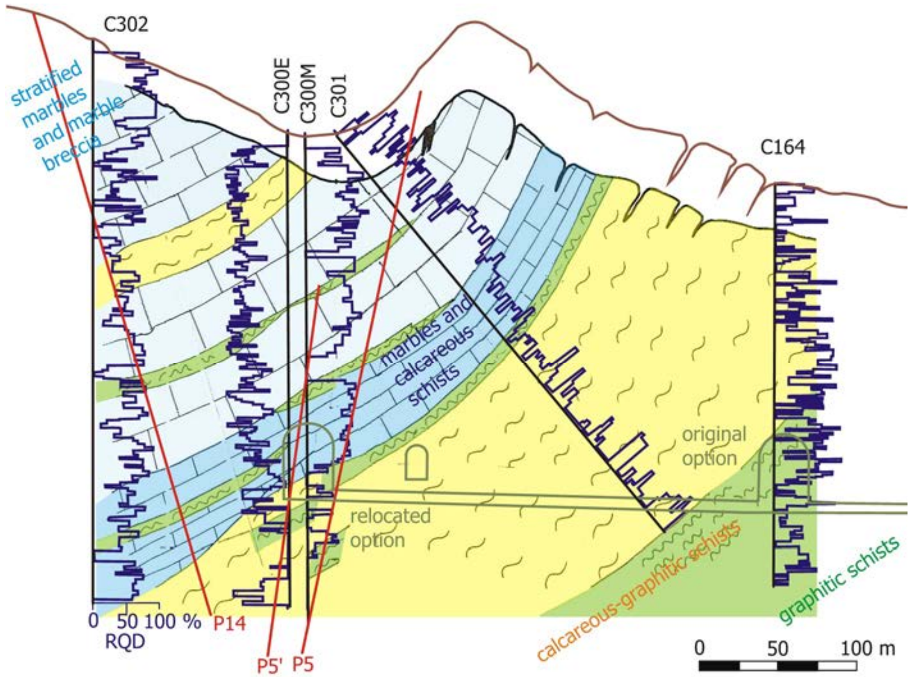


Fig. 8.5.11 Relocation of the cavern of a powerhouse

Survey work showed that the engineering-geological conditions were unsuitable for the construction of a pumped storage hydroelectric plant by placing a powerhouse and penstocks underground and therefore all subsequent work was directed towards identifying a suitable site for the powerhouse and penstocks on the surface. The site chosen for the powerhouse was located on a quasi-homogeneous block of crystalline calcareous schist of the Cobrito Formation; quartzitic schist of the Loma La Gloria Formation was also encountered during the survey of the site, but they did not crop out in the excavation. The survey demonstrated that the slopes of the excavation for the future powerhouse would be stable to a depth of 30 metres. The surface penstocks for the planned powerhouse were placed outside the areas affected by landslides.

The engineering-geological survey for the Centro Cuba PSHEP was certainly one of the most demanding contracts undertaken overseas by Czechoslovak engineering geologists at the end of the last century. This is demonstrated by the scope of survey work that was undertaken (Chap. 4.5) and by the details of the geotechnical survey. A total of 1,435 rock samples were tested in the laboratory, 17 loading tests and 15 shear tests in frames were made in the area of the upper reservoir, and other rock mechanics tests were deliberately carried out in the field on the sites of the foundations for other facilities. At the stage of the engineering project, attention was chiefly focused on the creation of a tectonic model for the sites of different objects and the determination of geotechnical design parameters for individual types of rocks and quasi-homogeneous blocks. Based on the correlations obtained between different physical-mechanical parameters and on direct laboratory measurements and field tests, geotechnical tables were compiled that were generally valid across the area of interest in which construction was to take place. These tables give geotechnical values for environment “a”, which is sound rock below the level of partial weathering and loosening of the rock mass, and for environment “b” corresponding to heavily weathered rock. Moduli of elasticity, deformation and shear strengths are defined relative to orientation either perpendicular or parallel to foliation. Based on the application of all the measured geotechnical and physical parameters and the correlations between them, it was possible to make a comprehensive geotechnical classification of rocks and of the rock mass for all the different lithologies underlying the various facilities of the hydroelectric plant so that the designer could be supplied with detailed data for input to the design of individual objects.

The following tables, the evaluation of physical and mechanical properties of rock mass (Tab. 8.5.9), physical and mechanical properties of rock matter (Tab. 8.5.10) and geotechnical classification of rock – rock mass (Tab. 8.5.11) serve as guidelines for the detailed planning of the geotechnical part of the survey and for the presentation of the results obtained. The complexity and size of the tables are proportional to the complexity of the geological structure of the area surveyed.

In conclusion, it must be emphasized that an important part of the geotechnical survey at all stages, but particularly during the period of construction and after a hydroelectric plant is commissioned, is the monitoring of the rock environment and the structure of the buildings. The aim is to verify that the specifications established during the geotechnical survey have been met, and that the facilities are stable and safe. On the one hand, this concerns monitoring during construction and after completion (Chap. 7.4.3), and the monitoring of active deformation or changes in the state of stress and loading on the other. In this case, geophysical methods of monitoring are often applied, for example to measure the effectiveness of a grout curtain, to determine seepage paths through a dam body or to monitor movements on slopes.

In common with other scientific disciplines, geotechnics has undergone dramatic changes recently. These do not relate so much to the discovery of new principles but to the improvements that have been made in the equipment used for testing and measurement and in the procedures used to record, process and present results. These changes have been made possible by developments in computer technology that have opened new possibilities for the control of tests, the processing of results and the creation of sophisticated mathematical and physical models. All these advances have enabled better understanding of the rock mass, its description and the prediction of its behaviour. Today, it is possible to monitor the behaviour of the rock mass continuously over time and the capabilities for making measurements of different variables are almost unlimited. The fact is that we are now able to detect phenomena for which we do not yet have a satisfactory theoretical explanation. The sophisticated technology now being introduced for survey and monitoring purposes is opening new perspectives that will require deeper theoretical investigation and encourage fresh approaches to the interpretation of the observed facts.

Table 8.5.9 Physical and mechanical properties

Formation: Zone	Subzone	Characteristic lithological type of rock	Numerical symbol: In geological sections	Numerical symbol: In logging graphs	Medium	Rock mass: Prtodyakonov	Poisson's ratio	Velocity of longitudinal waves	Resistivity	Seismic modulus of elasticity	Static modulus: Orientation to foliation	Static modulus of elasticity		Static modulus of deformation		Shear strength: Orientation to foliation	Cohesion		Angle of internal friction	
								km/s	Wm	GPa		GPa	GPa	GPa	GPa		MPa	MPa	°	°
Chispa II		Micaceous quartzitic schist	1	9	a		0.30	4.0–4.5	50–200	0.8	Undiffer-entiated	0.15		0.08		Undiffer-entiated	0.02–0.05		32	
		Graphitic quartzitic schist	2		b		0.35		25–50											
Cobrito III	1	Breccia marble	3	3	a	6	0.25	6.0	10–20	53	Undiffer-entiated	6.5		4.2						
					b	4	0.30	5.0												
		Stratified marbles, massive marbles	4	1	a	5	0.25	6.0	500–1000	53	Undiffer-entiated	7.0		4.7		Perpendicular to foliation / Parallel to foliation	1.0	0,3–0,5	45	40
					b	4	0.30	5.0				3.0		2.2			0.5	0,1–0,2	38	35
	2	Schistose crystalline limestones, weakly graphitic calcareous schists	5	2–2a	a	3	0.25	5.2	100–500	45	Perpendicular to foliation / Parallel to foliation	1.8			6.0		0,4	0,05	50	38
					b	2	0.30	4.5				1.5			2.0		0,2	0,03	38	34
	3	Graphitic calcareous schists to strongly graphitic calcareous schists	6	2b–2b'	a	1,5	0.25	4.7	50–100	43		0.7			4.7		0,2	0,05	40	36
					b	0,8	0.30	4.0	10–50			0.6			1.5		0,1	0,02	35	33
	4	Intensely folded graphitic calcareous schists	6	2c	a	1,5	0.30	3.0	cca 10	14		0.5			1.2		0,1	0,00	38	32
					b	0,4	0.35	1.8				0.4			0.8		0,05	0,05	28	28
	IP	Calcareous chloritic schists, micaceous calcareous schists	7	2d, 2f, 2e, 5	a	3	0.25	4.6	100–150	40		1.6			5.5		(0,5)	0,15	42	34
					b	2	0.25	3.9	20–100			1.2			1.6		0,1	0,05	35	30
Loma La Gloria IV		Micaceous quartzitic schists micaschists, micaschist gneisses	8	6–7	a	3	0.25	3.5–4.5	300–800	40		1.6			5.5		0,4	0,10	40	34
					b	2	0.32	2.0–3.5				1.2			1.6		0,1	0,00	35	30
	IP	Chloritic schists	10	4, 2f	a	33	0.30	5.0	100–300	42		3.3		3.0			0,4	0,05	50	38
					b	22	0.32	4.0	10–100			0.7		0.7			0,2	0,03	38	34
	IP	Talc schists	12	12	a	1	0.30		20–40	0.8	Undiffer-entiated	0.2		0.10			0.05	0,00	28	22
					b	0,6	0.40													
Metabasites and serpentinites	1	Metabasites of Loma Pedrosa type, metabasites of another type	11, 12	11	a	8	0.30	6.5	200–300	60	Undiffer-entiated	7.0		4.0		Undiffer-entiated	0.2		45	
					b	6	0.32	5.0				0.2		0.1			0.02–0.05		28	
	2	Serpentinites		11	a	6	0.30	6.0		52	Undiffer-entiated	4.0		2.5		Undifferenti-ated	0.2		40	
					b	4	0.35	4.5				0.2		0.1			0.02–0.05		25	

Table 8.5.10 Physical and mechanical properties – 1I

Formation: Zone	Formation: Subzone	Characteristic lithological type of rock	Numerical symbol: In geological sections	Numerical symbol: In logging graphs	Medium	Rock matter: Density	Bulk density	Unconfined compression strength, foliation of 45°	Splitting tensile strength on foliation	Coefficient of softening	Coefficient of Protodyakonov	Poisson's ratio	Velocity of longitudinal waves	Resistivity	Magnetic susceptibility	Dose intensity of radiation	Seismic modulus of elasticity	Static modulus: Orientation to foliation	Static modulus: Elasticity		Static modulus: Deformation	Shear strength: Orientation to foliation	Shear strength: Cohesion		Shear strength: Angle of internal friction	
						kgm^{-3}	kgm^{-3}	MPa	MPa				kms^{-1}	Ωm	10^{-6} SI	ppm	GPa		GPa		GPa					
Chispa II		Micaceous quartzitic schist	1	9	a	2540	2448	19	4.0	0.6	1.8	0.35	5.4	480	761	0.35	61	Perpendicular to foliation; Parallel to foliation	35		32	Perpendicular to foliation; Parallel to foliation				
		Graphitic quartzitic schist	2		b		509						4.5	170	250	0.15										
Cobrito III	1	Breccia marble	3	3	a		2697				5.0	0.25	5.95	433	19	3.7	70		51		51					
					b		42						0.26	500	30	2.9										
		Stratified marbles, massive marbles	4	1	a	2709	2669	38.2	4.7	0.8	4.0	0.25	6.08	7200	150	0.67	74		54	74			1.2	0.8	52	40
					b								0.49	5400	195	1.0			59		55		0.7	0.3	45	38
	2	Schistose crystalline limestones, weakly graphitic calcareous schists	5	2–2a	a	2735	2682	20.5	3.8	0.8	2.0	0.25	5.79	1521	41	4.96	66		30	70			0.7	0.25	45	38
					b								0.37	1980		2.64			44		40		0.3	0.08	38	32
	3	Graphitic calcareous schists to strongly graphitic calcareous schists	6	2b-2b‘	a	2731	2665	7.3	1.5	0.7	0.8	0.25 – 0.27	5.65	205	73	14.1	63		17	44			0.4	0.12	40	38
					b								0.40	259		26.5			31		29		0.1	0.05	38	33
	4	Intensely folded graphitic calcareous schists	6	2c	a	2730	2642	4.0	0.8	0.5	0.4	0.30	3.60	20	70	6.1	26		10		8		0.15	0.05	38	32
					b								0.50			4.0							0.05	0	32	28
	IP	Calcareous chloritic schists, micaceous calcareous schists	7	2d. 2f. 2e. 5	a	2909	2852	19.0	4.0	0.7	1.9	0.27	5.59	2170	373	4.2	62		16	57			0.5	0.2	42	35
					b								0.56	1570		2.8			36		33		0.05	0	35	32
Loma La Gloria IV		Micaceous quartzitic schists, micaschists, micaschist gneisses	8	6–7	a	2808	2742	19.0	4.1	0.7	1.9	0.25	5.90	650	3950	0.9	68									
					b																					
	IP	Chloritic schists	10	4. 2f	a		2813				1.9	0.30	5.59			5.3	61									
					b		71						0.49			9.9										
	IP	Talc schists	24	12	N						0.6	0.23														
Metabasites and serpentinites	1	Metabasites of Loma Pedrosa type, metabasites of another type	11, 12	11	a		3084				6–8	0.25	6.54	2695	823	0.17	86									
					b		57						0.26	2383	146	0.20										
	2	Serpentinites		11	a												86									
					b																					

Explanation for Tables 8.5.9 – 8.5.11

Values in green state by how much it is necessary to decrease the value for the weathered environment

Values of moduli are given for the orientation to foliation from 20 to 90°

Values in blue are estimated

Table 8.5.11 Geotechnical properties

Formation		Characteristic lithological type of rock	Numerical symbol		Medium	Geotechnical classification of rocks – rock mass									
						ČSN 73 1001 class		Protodyakonov fp		Barton (NGI) Qa		Deere RQD		Workability ČSN 73 3050	Driveability
Zone	Subzone		In geological sections	In logging graphs		No.	Jointing	No.	In words	No.	In words	%	In words	Class	Degree
Chispa II		Micaceous quartzitic schist	1	9	a	2a	high	1.8	fairly soft	0.08	completely extremely bad	0	very bad	4	II
		Graphitic quartzitic schist	2		b	3a	high	0.1	mushy	0.03	completely extremely bad	0	very bad		III
Cobrito III	1	Breccia marble	3	3	a	1a	low	5	fairly hard	30	good	55–70	worse	6	I
					b	2a	medium	2	fairly soft	4	sufficient	30–50	bad		
		Stratified marble, massive marble	4	1	a	1a	low	4	fairly hard	20	good	55–75	bad to worse	6	I
					b	2a	medium	2	fairly soft	3	bad	30–50	bad		
	2	Schistose crystalline limestone, weakly graphitic calcareous schist	5	2–2a	a	1a	medium	2	fairly soft	5	sufficient	40–70	very bad to bad	6	I
					b	2a	high	1	soft	1	bad	20–40	bad		
	3	Graphitic calcareous schist to strongly graphitic calcareous schist	6	2b–2b‘	a	2a	high	0.8	soft	1.5	bad	10–40	very bad to bad	5	I
					b	3a	high	0.5	loose	0.1	extremely bad	0–10	very bad		II
	4	Intensely folded graphitic calcareous schist	6	2c	a	2a	high	0.4	loose	0.1	extremely bad	0–25	very bad	4	II
					b	3a	high	0.2	fairly soft	0.02	completely extremely bad	0–10	very bad		III
	IP	Calcareous chloritic schist, micaceous calcareous schist	7	2d, 2f, 2e, 5	a	2a	medium	1.9	fairly soft	1.5	bad	50–70	bad to worse	5–6	I
					b	3a	medium	0.5	loose	0.05	extremely bad	30	bad		II
Loma La Gloria IV		Micaceous quartzitic schist, micaschist, micaschist gneiss	8	6–7	a	2a	medium	1.9	fairly soft	20	good	60–85	very bad to bad	5–6	I
					b	3a	medium	0.2	mushy	0.05	extremely bad	0–20	very bad		II
	IP	Chloritic schist	10	4, 2f	a	2a	medium	1.9	fairly soft	1.5	bad	50–70	bad to worse	5–6	I
					b	3a	high	0.2	fairly soft	0.5	very bad	30	bad		II
	IP	Talc schist	21	12	N	7	high	0.6	very soft	0.01	completely extremely bad	0	very bad	4	III
								0.2	mushy						
Metabasite and serpentine	1	Metabasite of Loma Pedrosa type, metabasite of another type	11, 12	11	a	1a	low	8	hard	150	very good	70–90	bad	6	I
					b	2a	low	6	fairly hard	20	good	30–50	bad		
	2	Serpentine		11	a	1a	low	6	fairly hard	100	very good	55–70	bad to worse	6	I
					b	2a	medium	4	medium	10	good	30–50	bad		

9. Engineering-Geological Survey of the Reservoir Areas

The engineering-geological survey of the reservoir areas of dams is usually concerned with questions relating to the watertightness of the floor and sides of reservoirs, to the stability of the banks of the future reservoir and to the possibility that a reservoir may silt up with bed load sediments. A part of the survey will also be concerned with the evaluation of the changes in the environment likely to be caused by the future water reservoir and, commonly, surveys must be made of archaeological remains, biodiversity and historical sites. An important aspect of the survey will be the evaluation of the impact of the maximum level of the groundwater table on the surrounding slopes, because any change in the hydrological regime caused specifically by the rise of the groundwater table or by frequent fluctuations in the water table due to variations in the water level in the reservoir, can have important effects on the stability of slopes and on the exploitation of resources of drinking or industrial water. Also, devaluation of mineral deposits after flooding must also be taken into account. It is also important to carry out a survey of the reservoir area and the adjacent slopes for construction material suitable for opening borrow pits or stone quarries to supply material for construction of the dam. The advantages of extracting construction materials from the reservoir area are their proximity to the site where they are needed for construction and also that, at the same time, the space available for water storage behind the dam is increased without incurring significant costs for the restoration of the excavations.

The factors governing the impermeability of the bottom and slopes of a future reservoir have been discussed in Chapter 5. Some examples of seepage into the sides of reservoirs and through the rocks at sites where dams are keyed are also given in Chapters 2.3.1 and 2.3.2. In Chapter 2.3.2, the stability of the sides and slopes of future reservoirs has been discussed using a number of practical examples, and also in Chapter 4.4. The evaluation of the impact on the environment caused by reservoirs is the topic of Chapter 2.3.6.

The stability of the banks of a future reservoir is governed by two main factors. The first of these is that flooding changes the stability of slopes. The second important process that threatens stability of the banks is pounding by waves, which results in the formation of beach platforms and cliffs by erosion and, in unfavourable conditions, this can cause landslides.

After the reservoir behind a designed dam has filled up, the saturated rocks underwater are lightened because of Archimedes' law. The lightening, mainly of the soil mass, will have the effect of reducing the weight on parts of the slopes formed by soils and fractured rocks, in particular on areas of slope failure, thus reducing their stability. Above the water surface, infiltrated surface water flows away more slowly due to the reduction of the hydraulic head, and thus the mass of saturated soil increases and the consistency changes causing decreases in strength parameters. In areas of dormant or incipient slope failure the stability is further reduced. When rapid changes in the water level in the reservoir take place, the hydraulic gradients are created in the soil water so that flow takes place and the weight of soil changes rapidly. These changes in weight can also initiate deformations or trigger movement in dormant landslides.

Bearing in mind what has been discussed elsewhere in this book, the main subject for discussion in this chapter will be the modification of reservoir banks by abrasion, erosion, suffosion and other factors, and the various types of slope failures that affect them. Slope failures can take place because of the rejuvenation of movements in dormant landslides or because the fronts of existing deformations are lightened by immersion or weakened by saturation and in other cases new slope failures can be triggered by changes in the configuration of reservoir banks. These problems have all been faced in the various projects that Geotest has undertaken in the past.

9.1 Modification of Reservoir Banks

Up until the 1950s, relatively little attention had been given to the factors governing changes in banks surrounding water reservoirs. The interest of water managers was focused particularly on constructing dams in topographically favourable settings, particularly in deeply incised valleys where the stability of banks presented few problems. The favourable and mostly trouble-free dam sites, however, were quickly exploited and it was necessary to identify new sites in wide, open valleys where, especially in the reservoir area, problems of instability on banks and with reservoir silting soon became apparent.

Generally, the stability and erosion of reservoir banks should be considered before the construction of a dam takes place, i.e. at the stage when a hydro-engineering structure is being designed. Because, in the past, attention had not been paid to these issues, for example at the Orava, Rožnów, and Charvak dams, the problems arising because of changes affecting the reservoir banks and the prediction of how they would develop were sometimes solved after the dam had been put into operation. From this point of view, the research project “Bank Changes in the Reservoir Areas of Dams”, undertaken in the former Czechoslovakia during the last century yielded some very valuable results.

The proper evaluation of the geodynamic processes caused by the construction of a dam and the prediction of how they will progress once the reservoir is in operation are important tasks for the engineering geologist. The survey of these processes is usually divided into two stages, preliminary and detailed. The step-by-step procedure and the scope of the survey are governed by the design of the hydro-engineering structure and the stage of preparation as well as by the complexity of topographic, geological, climatic and other factors that have an effect on the site, including the population density, the location of industrial plant, the need to protect certain important archaeological, historical or natural objects and the time allocated for completion of the project. Survey work must be concerned not only with the immediate area of the future water reservoir, but also with the geological setting of the wider surroundings of the site of interest. This involves the engineering-geological mapping of the reservoir area and its adjacent slopes and the engineering-geological zoning of the banks. To carry out this task effectively, a good knowledge of geological conditions in the wider area around the dam and reservoir is absolutely essential.

According to the agents responsible, geodynamic phenomena can be classified into natural and anthropogenic categories. Natural phenomena are subdivided into endogene and exogene categories. For the purposes of this evaluation, if endogene processes including deformation of the Earth’s crust caused by earthquakes and volcanic activity and the effects of anthropogenic activity are neglected, then the exogene processes that have the most important influence on the reservoir areas of dams are abrasion and water erosion, weathering processes, mechanical suffosion and slope movements. It is natural that attention should also be given to geodynamic processes at the dam site, especially during the period of survey and construction. After putting the hydro-engineering project into operation, these will play a significant role, especially in the reservoir area.

Local structural and climatic conditions, together with processes of erosion/abrasion are considered to be the main factors acting in the dam reservoir areas. Structural conditions are determined by geological, hydrogeological and topographic factors and by the intrinsic physical-mechanical properties of the rocks. These natural conditions cannot be influenced or changed to any substantial degree. Also, climatic conditions, which include the effects of rain and snow precipitation, the effect of temperature and wind, cannot be changed. These are chiefly responsible for changes in the physical-mechanical properties of rocks and soils. The wind causes pounding by waves and this in turn is the cause of abrasion. Their contribution to the bank-forming process must be assessed by thorough analysis of local conditions (Fig. 9.1.1).

Fig. 9.1.1 Co-action of erosion and abrasion (a photo by O. Horský - 1968)

In the bank zone of the reservoir, erosion and abrasion will have a strong effect. Where the surface of the water laps steadily against the banks of the reservoir, an abrasion platform is eroded at a certain level in the profile of the bank. The width and steepness of the platform depend on the slope and height of the bank, the height of waves and their energy and particularly on the rocks in which the platform is being cut. The shape of the platform will progressively evolve depending on the local conditions, and the changes caused by pounding waves and other effects on the banks will diminish. The equilibrium underwater profile of the abrasion platform is also established by the action of waves on the accumulation of eroded material. The formation of an equilibrium profile, however, will only take place in conditions where the level of the water surface does not change dramatically.

When evaluating reservoir areas, engineering-geological zones are defined in terms of the action of the water reservoir against the banks and valley slopes. For this purpose, the perimeter of the reservoir area is divided into sections in which the development of the bank zone is predicted to be similar. In essence, two zones on the reservoir perimeter are distinguished, namely the zone of the bank threatened by the dynamic effects of the reservoir on the natural environment, and the unthreatened zone. These zones are subdivided into sub-zones according to the types of processes that will cause degradation of the banks. These include zones subject to erosion, erosion and slip, erosion and accumulation, rock falls, unstable scree, landslides and other types of banks. The banks in the unthreatened zone are mostly rocky cliffs, very flat banks composed of soils resistant to erosion by water, or sections of the bank where the destructive impacts of the water surface are minimal, such as certain bays, the far ends of the reservoir or sections that have been artificially reinforced. The engineering-geological zoning of the banks of a reservoir is, so to speak, the final task in the survey of the reservoir area; it is very important because it provides information about the impact of the water reservoir on the natural environment and indicates where technical adjustment or reinforcement of the banks may be needed.

Analysis of factors and conditions governing the degradation of the banks of certain water reservoirs in Bulgaria, the Czech Republic and Slovakia shows that regional geological factors play the dominant role in modification of the banks. The important part played by erosion/abrasion has also been demonstrated. Nevertheless, for the purpose of predicting the recession of the shorelines of reservoirs and the proposal of remedial measures, it is necessary to take account of factors other than erosion alone.

Table 9.1.1 Intensity of development exogene processes in loamy sediments

Exogenous process	**Dam reservoir**					
	Orava	**Iskar**	**Kirdzali**	**Ivaylovgrad**	**Topolnitsa**	**Nechranice**
Weathering	++	+++	+++	–	+++	+++
Abrasion	+++	+++	+++	++	++	++
Accumulation	+	–	++	+	++	+
Slope erosion	++	++	+++	++	++	+
Suffosion	++	+	–	–	–	+
Landslide	+++	+	+	+	++	++
Screes, rock falls	+++	++	++	+++	+++	++
Effect of ice	++	+	–	–	–	+

Deployment degree of exogenous process			
+++	Significant	+	Little significant
++	Medium significant	–	Insignificant or nothing

The diagrams and photographs below show the typical effects of exogene processes on silty clayey and clayey sediments that cause the destruction of banks around water reservoirs. These processes are listed in Table 9.1.1, in which examples from water reservoirs in Slovakia (Orava), in the Czech Republic (Nechranice) and Bulgaria (the remaining reservoirs) are given.

Weathering is very intense, particularly in areas with an arid climate, where precipitation is minimal (Nechranice). Erosion takes place around all reservoirs and is most intense in reservoirs with banks of sandy silty clay and loess. The stronger the gusts of wind and the greater the movement of the surface water is, the greater the rate of erosion. Disintegrated materials are accumulated in lower parts of reservoirs and in bays. Slope erosion prevails, for instance, in Bulgarian reservoirs, but also occurs in Orava (Fig. 9.1.2).

Fig. 9.1.2 Erosion of accumulations below abrasion cliffs (a photo by O. Horský - 1968)

Processes of suffosion are developed mainly in re-washed loess and in eluvial deposits on the Neogene. A typical example is the Orava reservoir, partly also the Nechranice reservoir. Landslides are typical on the banks of the Orava reservoir (Fig. 9.1.3), also locally in Bulgarian reservoirs and in the Nechranice reservoir (Fig. 9.1.4). In some reservoirs, slope failures reach considerable dimensions. Some examples have already been given in the preceding chapters (Vajont, Fig. 2.3.31, Mingchukur, Fig. 4.4.4).

Fig. 9.1.3 Landslides on the banks of the Orava reservoir (photos by O. Horský - 1968)

The collapse of talus and falls of rock are examples of mass wasting on the banks around reservoirs. The erosion of banks by ice is characteristic of reservoirs where the winters are long and cold, as is the case at Orava. The Nechranice reservoir is also affected by ice, but to a lesser degree. The exogene processes described all have a greater or lesser effect on the banks of the water reservoirs listed in Table 9.1.1. The differences in the impact of these exogene processes on individual reservoirs are determined by local climatic factors.

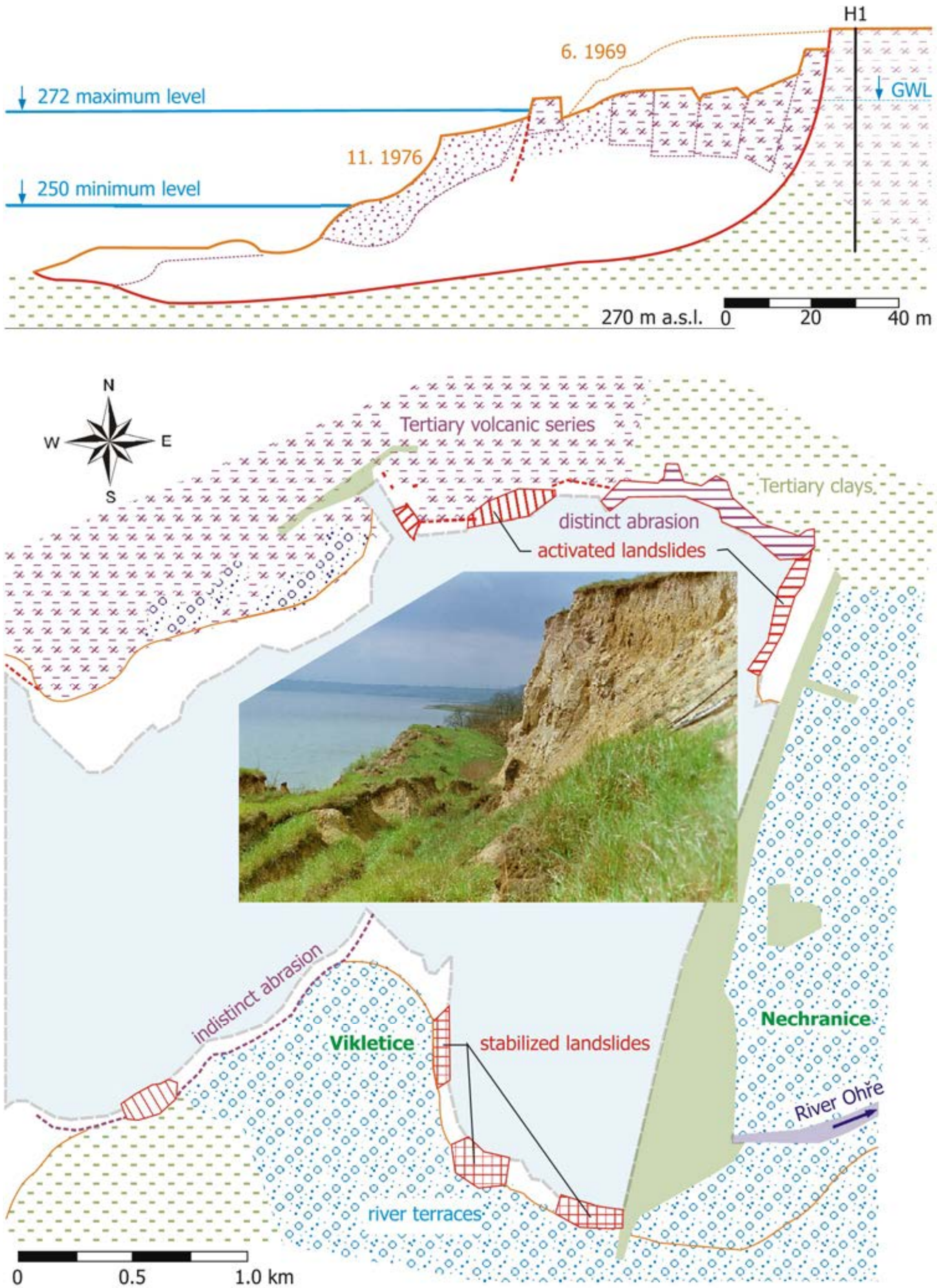


Fig. 9.1.4 Degradation of banks on the Nechranice reservoir (after Rybář, 1977 - a photo by O. Horský - 1972)

Because the reservoirs chosen as examples have all been in operation for 15 to 35 years and the state of their banks has been monitored more or less systematically, a valuable record of the interaction of the separate exogene processes causing the erosion of reservoir banks and the changes that have taken place now exists. This can be used as a basis for predicting the behaviour of the banks of water reservoirs in other areas. The characteristics of some selected reservoirs are given in Table 9.1.2.

Table 9.1.2: Characteristics of selected dam reservoirs and their banks

Dam reservoir	River	1 [m]	2 [km²]	3 [km]	4 [m]	5 [m]	Geology
Nechranice	Ohře	200–300	13.5	20	48	22	Clayey sediments of Tertiary and Quaternary
Orava	Orava	550–650	35.0	70	40	15	Palaeogene and Neogene flysch (claystone)
Šance	Ostravice	500–600	3.0	7	50	36	Cretaceous flysch
Žermanice	Lučina	200–300	2.5	15	47	15	Clay, claystone
Olešná	Olešná	300	0.7	5	16	2	Palaeogene flysch, re-washed loess
Rozkoš	Úpa	200–300	10.0	17	26	3	Cretaceous sediment
Hracholusky	Mže	300–400	4.6	36	34	14	Phyllite, metamorphic rock
Iskar	Iskar	600–950	30.0	35	70	28	Granite, metamorphic rock
Ivaylovgrad	Arda	200	23.0	73	72	16	Phyllite
Tsonevo	Kamchiya	200–600	17.3	63	55	25	Limestone, clayey sediment
Kirdzali	Arda	200–600	16.1	85	101	36	Metamorphic rock
Topolnitsa	Topolnitsa	200–600	5.7	35	65	27	Metamorphic rock
Koprinka	Tundzha	200	11.2	38	44	13	Granite
Andijan	Karadarya	790–910	56.0	47	120	120	Palaeozoic igneous rock, siltstone
Angat	Angat	80–300	23.8	160	125	50	Tertiary volcanic rock

Explanatory notes:

1	*Height conditions in reservoir area*
2	*Area of water surface*
3	*Perimeter of banks*
4	*Maximum depth*
5	*Maximum amplitude of fluctuation*

In the geological description, solid rocks also include the rock mantle

The Czech and Slovak reservoirs are amply documented and continue to be a focus of interest for water managers and researchers, however the Bulgarian reservoirs at Iskar, Ivaylovgrad, Tsonevo, Kardzhali, Topolnitsa and Koprinka are also worthy of mention. Research has demonstrated that the Balkan Peninsula and the wider Carpathian-Balkan Engineering-Geological Region is characterized by factors leading to particularly aggressive degradation of the banks of water reservoirs. The processes causing erosion of the banks of Bulgarian reservoirs are typically linear, with cliffs of different heights and beach platforms with stepped profiles. As in the case of Czech reservoirs, mass wasting by cliff falls and landslides also occurs locally. These phenomena are chiefly the result of erosion and degradation in the zone of water level fluctuation. The net result is the transport of detritus that accumulates as a bar below the water surface. The banks formed around the Bulgarian reservoirs are mainly of erosion, erosion-accumulation, landslide and also, in contrast to Czech reservoirs, of swampy type.

The Orava reservoir is an example of a reservoir where changes in the banks required special technical measures and, locally, costly remediation (Fig. 9.1.5). The Orava reservoir is one of the largest water reservoirs in Slovakia in terms of its water area and backwater volume. With a maximum backwater level at an elevation of 603.0 m a.s.l., the water covers an area of 32.8 km^2, and the volume of the reservoir is 331 million cubic metres, the total length of the banks being 70 km (Fig. 9.1.6).

Fig. 9.1.5 Strengthening of banks with stone packs (a photo by L. Lincer - 1992)

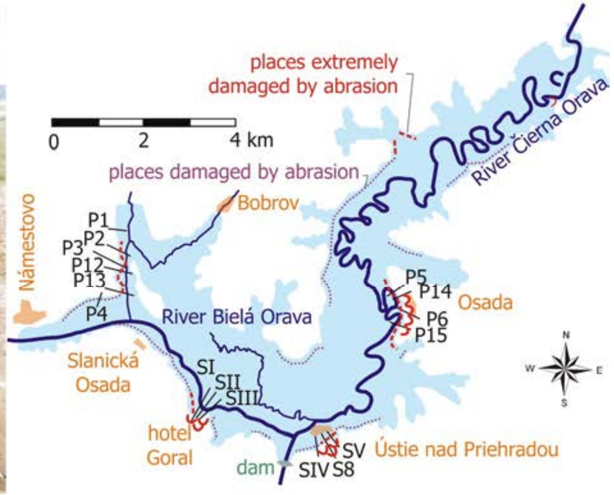


Fig. 9.1.6 Plan of the Orava reservoir

The construction of the dam has had a favourable effect on the development of the whole area. The water reservoir has become a much sought-after centre for recreation. Around its banks, however, the operational fluctuation of the water level, pounding by waves and other exogene processes have had a marked effect. After fifteen years of operation, in some places the banks of the reservoir had receded to a distance of over 40 metres and cliffs, locally up to 20 metres high, had been eroded. On certain sections of the shoreline, extensive landslides occurred and the destructive effects extended up to 160 metres inland from the shoreline (Fig. 9.1.7).

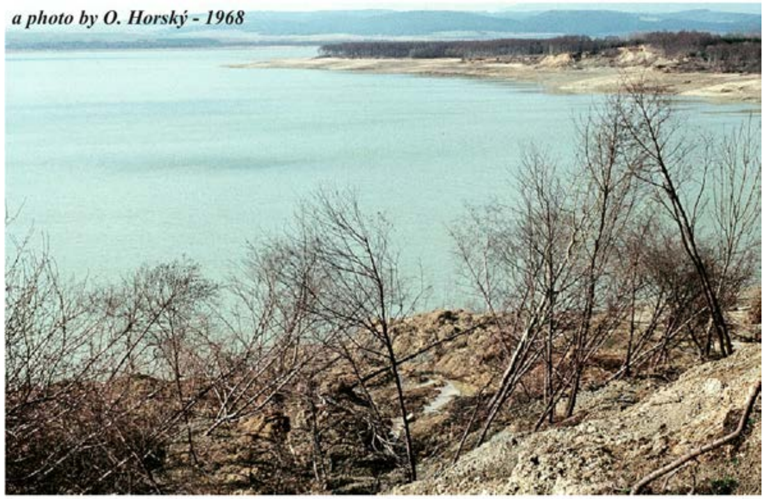


Fig. 9.1.7 Recession of the shoreline

The recession of the banks threatened some recreational facilities, and also areas of valuable agricultural land. As a result, in many places, it was necessary to restrict the recreational use of beaches and the adjacent banks. The eroded and slipped material was washed out into the reservoir where the water was made muddy by waves and the bottom of the reservoir progressively silted up.

Despite the fact that a careful investigation of the complicated geological conditions in the Magura Flysch was carried out after World War II to prepare for the construction of the dam of the Orava reservoir, the importance of making an assessment of the behaviour of the banks was not recognized. The dam was brought into operation in 1954 and, after a few years of operation, the detrimental effects of erosion were already evident. It became clear that the importance of investigating the dynamic effects of erosion on the shoreline had been underestimated. When erosion and landslides began to threaten some recreational facilities and a marina, a detailed engineering-geological survey was commissioned in 1967. The aim was to evaluate the impact of the reservoir on the surrounding banks and to make proposals for their protection in the most vulnerable places. Due to the area of the water in the reservoir and the degree to which the banks were affected, this task demanded a comprehensive investigation of the geological, topographic and hydrological conditions. An engineering-geological map at a scale of 1:10 000 was compiled to cover the entire periphery of the reservoir. A map of engineering-geological zones was compiled, in which the banks were classified according to the prevailing natural conditions and their predicted response to erosion so that remedial measures could be proposed. Special attention was given to a detailed geological and geotechnical description of the processes of sedimentary deposition in the area of cliffs; the measurement of the profiles of stable and unstable beaches, both above and below the water surface; the gradients of underwater bars; the slopes of talus and cliff profiles; zones depleted by landslides; and the shape and character of shear planes. These observations were essential to enable predictions about the future development of these features.

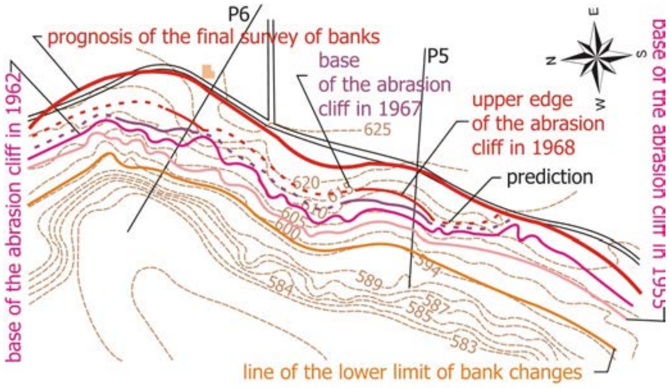


Fig. 9.1.8 Pattern of the recession of the base of a bank cliff

In the areas around the reservoir most affected by the impounded backwater, classical engineering-geological investigations were supplemented by observations of auger holes, pits and drill core so that the geological and hydrogeological conditions of the threatened bank could be described in greater detail. In two areas most affected by landslides, a geophysical survey was also carried out. The chief physical-mechanical properties of soils from shear planes, cliffs, beaches and sedimentary accumulations were also determined. At places exposed above water, transverse profiles were surveyed and standardized and profiles were then carried offshore by sonar measurements.

Fig. 9.1.9 Devastation of a marina (a photo by O. Horský - 1968)

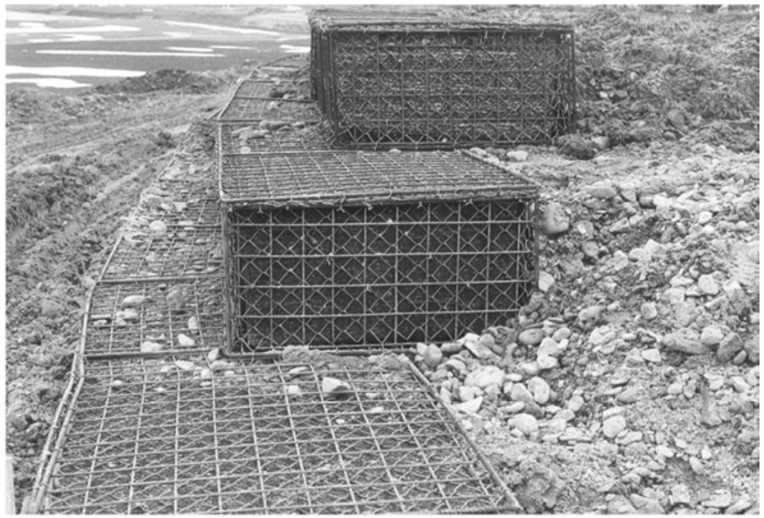


Fig. 9.1.10 New protection of banks by gabions (a photo by O. Horský - 1992)

Using profiles of the area made in 1953 (before the reservoir had been filled up), 1955 and 1962 and topographic maps at a scale of 1:5 000 (with contours at 1 to 5 m intervals), the scarps and toes of cliffs and water level lines were plotted photogrammetrically around the sites of standard profiles using stereoscopic pairs of aerial photographs (Fig. 9.1.8).

The aerial or remote sensing stereoscopic images and the geodetic surveys of the selected sites (1967 to

1968) enabled a very detailed evaluation of the recession and destruction of banks at the most affected places so that predictions could be made about their future development and their probable final state. On this basis, proposals for suitable biotechnical and technical interventions were made. Based on the results of the engineering-geological survey, successful remediation of the landslide area at the hotel Goral was carried out (Figs. 9.2.2 and 9.2.3). Also, the banks in the marina affected by degradation were reinforced (Fig. 9.1.9). To counteract the effects of erosion, measures were undertaken near the municipality of Námestovo at the Polhoranka site using gabions and revegetation (Fig. 9.1.10).

In 1983, escalating demands for environmental protection provided the impetus for a re-evaluation of changes affecting the banks at a number of selected sites. Due to the fact that fifteen years had elapsed since the last actual survey of the banks, it was possible to assess the correctness of the predictions about changes to the banks made in 1968 and to assess the efficiency of the remedial measures that had been carried out at that time. The predictions about the changes to the banks that were made were based on the local geological and hydrological conditions. Minimum and maximum extents of the predicted changes were estimated with reference to the range and frequency of the fluctuations in water level and the geological composition and structure of the bank.

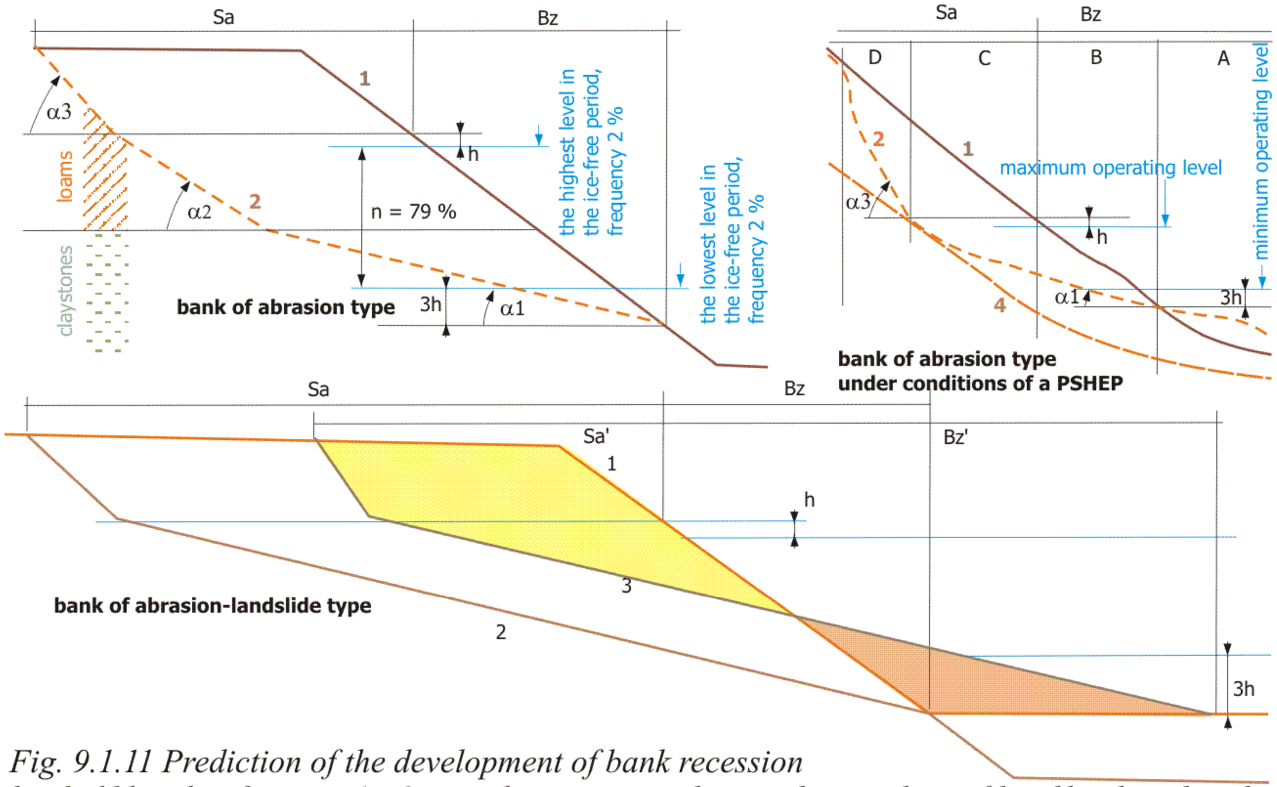


Fig. 9.1.11 Prediction of the development of bank recession
h – half-height of waves, 1 - 2 – angles corresponding to the steady profile of bank within the reach of abrasion, 3 – angle of the stable slope of an abrasion cliff, Sa – predicted recession of bank above the maximum water level + h/m (Sa' – the same for a bank of abrasion-landslide type), Bz – range of bank changes below the maximum water level + h/m (Bz' – the same for bank of the abrasion-landslide type), A – area of depositional processes and possible movement of detritus, B – zone of abrasion phenomena in the range of level fluctuation, zone in which changes in hydrostatic and hydrodynamic pressures have an impact on bank stability, C – zone of abrasion phenomena at the maximum operating level, D - zone within which geodynamic processes are active, 1 – original ground, 2 – predicted consolidation of the bank profile after the end of abrasion, 3 – predicted consolidation of the bank profile after the end of abrasion in a bank of abrasion-landslide type, 4 – bedrock surface

In the Orava reservoir, banks formed by Neogene and Quaternary clayey and clayey-sandy sediments and in re-washed loess are strongly affected by erosion for 2 % of the time, sandy gravel for 3 % of the time and stony-loamy debris, mainly Palaeogene, for 5 to 8 % of the time. In the winter and spring seasons, when the water level drops by up to 15 metres, depending on the regulation of the reservoir, erosion takes place predominantly in Neogene clayey-sandy sediments. In order to determine the upper and lower limits of the zone affected by water erosion, it is necessary to add half the wave height (= “h”) to the upper limit of the zone reached by the water in the reservoir, and to subtract three half wave heights (= “3h”) from the lower limit. The wave height (= “2h”) for each profile must be chosen with regard to the depth of water in the reservoir, the wavelength and the wind velocity (Fig. 9.1.11).

The most frequent level at which the water stood during the first fifteen years of operation was 601.75 m a.s.l. (for 8 % of the time) and this has had a marked effect on the pattern of erosion. In later years, the distribution of the frequency of water levels became more even and smoother, and the proportion of time during which the water level stood at 601.75 m level was reduced to 5 % over thirty years; in 1991 the proportion was reduced still more to 3.5 % (Fig. 9.1.12). This level caused the intensive erosion of all sediments except for the solid Palaeogene rocks, and within the prevailing range of its action (600.55 to 602.15 m a.s.l.) beach platforms were created, migrating landwards as the banks eroded (Fig. 9.1.13). The prediction for the recession of the banks around the Orava reservoir under the conditions created by the operation of a PSHEP is shown in Figure 9.1.11.

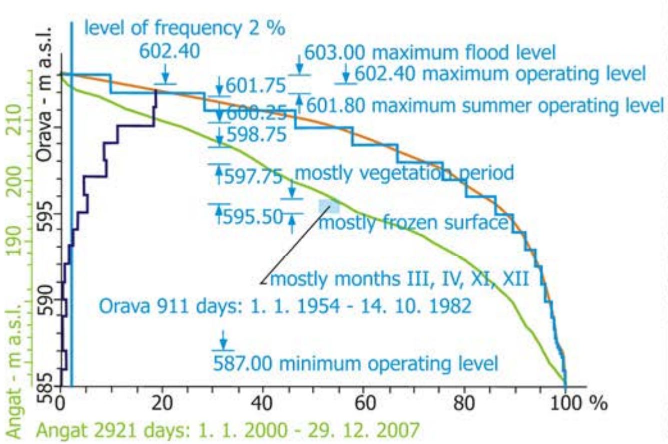


Fig. 9.1.12 Frequencies of levels on the Orava and Angat reservoirs

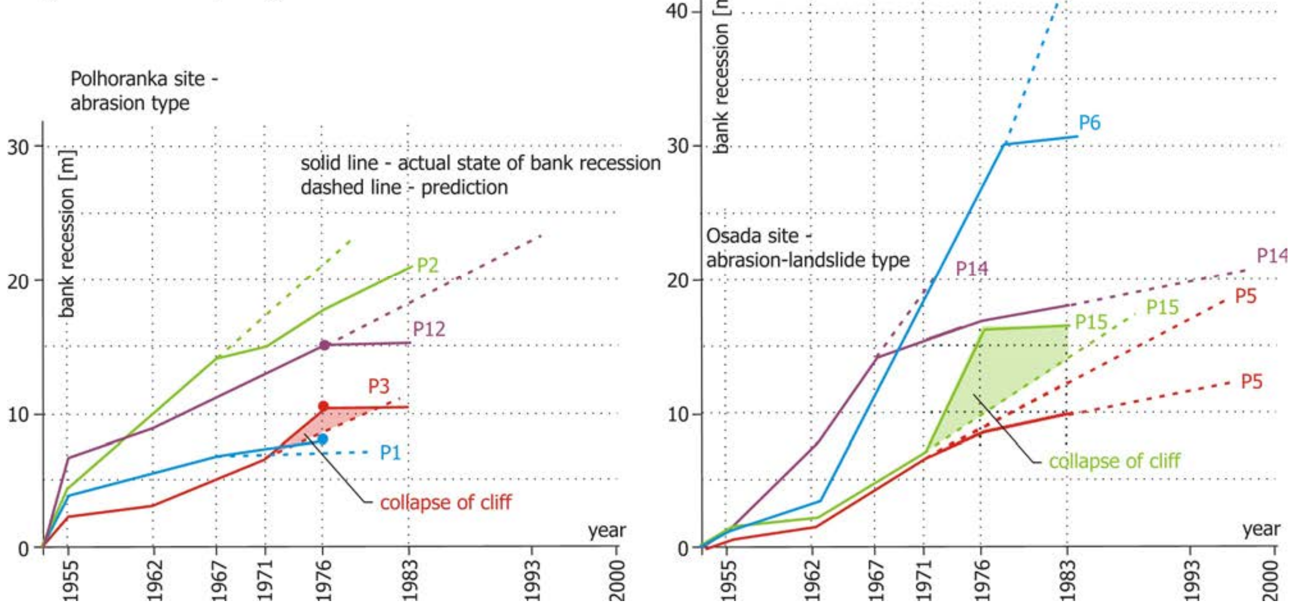
Fig. 9.1.13 Beach abrasion platforms (a photo by O. Horský - 1968)

In order to predict the future shapes of banks, an equilibrium profile for the appropriate soil was projected inland from the lower limit of the bank. Another morphological feature to be determined was the shape of that part of the bank lying above the upper limit of the equilibrium slope determined by the local geological conditions. The recession of the shore (Sa) is then the horizontal distance between the point of intersection of the upper limit of the change in the bank with the original topographic surface and the scarp of the cliff or the boundary of the zone of depletion by a landslide.

The progressive changes in the banks of the Orava water reservoir show that the processes of erosion in readily washable soils that lead to the formation of a more or less stable profile in the bank take 30 to 50 years from the time when the reservoir is filled up. The subsequent reworking of the steep cliffs created by this process takes a long time and the reservoir has little effect on this once the equilibrium profile has been achieved by erosion. Erosion of the banks of the reservoir will be reactivated if the water level drops significantly. The long-term drawdown of the reservoir will trigger the erosion of a new equilibrium profile in the exposed banks and in accumulations of detritus deposited underwater by rivers and erosion of the slopes and other exogene processes. In order to maintain a relatively stable shoreline once an equilibrium profile has been established, it is important that the water levels in the reservoir are regulated, as far as possible, within the limits that governed the erosion of that profile.

In the case of banks produced by erosion and accumulation, the rate of erosion is highest during the first two to three years and subsequently diminishes in a linear pattern (Fig. 9.1.14). In banks of erosion-landslide type, the rate of erosion is again highest during the first two to three years, later the rate is diminished by the formation of high cliffs. After about eight to ten years, the stability of these high cliffs is gradually reduced and they begin to collapse. The collapsed detritus is easily broken up by wave action and the material is washed out and redeposited, this exposes the bank to further erosion and the shoreline advances inland once again. This dynamic pattern of events leads to an acceleration in the rate of erosion and degradation of the banks.

Fig. 9.1.14 Graph of bank recession



The rate of erosion that governs the shape of the profile is directly related to the intensity of the exogene processes that cause it, namely weathering, leaching, and suffosion. The development of the slope is further related to the distribution of stress in the mass and the groundwater regime, i.e. factors that have an important effect on the stability of the slope after the reservoir is filled up. Examples of the recession of banks of erosion and abrasion-landslide type are given in Figure 9.1.15. At Polhoranka, the banks receded 26 metres in only the first 15 years of operation of the reservoir; at Osada the banks receded by 47.5 metres (1967), and in another 15 years by 76 metres (1982).

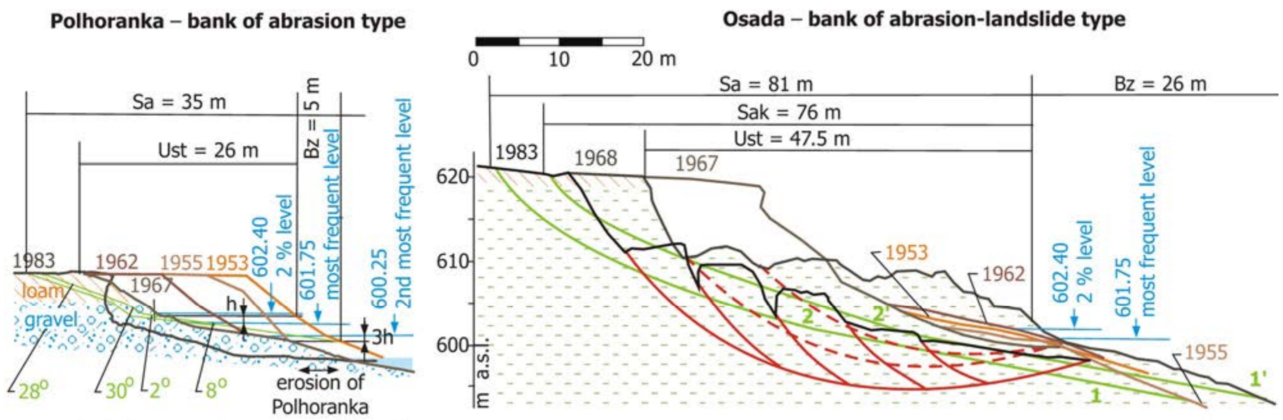


Fig. 9.1.15 Prediction of bank recession
(Sa – predicted bank recession for bank of abrasion type (m), Sak – predicted bank recession for a bank of abrasion-landslide type (m), Bz – range of bank changes below water level (m), Ust – bank recession as of data evaluation (m), 1, 2 – lower limit of bank changes for a bank of abrasion type (m), 1′, 2′ – displaced lower limit of bank changes for bank of abrasion-landslide type (m)

In contrast to relatively static water reservoirs, the banks of reservoirs supplying pumped storage hydroelectric plants are modified particularly by the rapid, mostly day-to-day, fluctuations of the water level over a wide range, and by the dynamic effects of wave action at the maximum and minimum operating levels. Due to these short-term and relatively large fluctuations of the water level, a greater amount of material will be disturbed. The banks of PSHEP reservoir will have a profile of erosion following the pattern of frequency of changes in the water level with two sharply delineated maxima at the highest and lowest operating levels. In predicting changes in banks caused by erosion, it is necessary to take account of this fact and to find a solution that will cope with both the maximum and minimum operating levels. If a PSHEP is built directly on a river (Dalešice PSHEP) and the fluctuation of the water level due to water pumping and re-pumping during the operation of the plant is minimal, then the predictions of changes in the banks can be made in the same way as for reservoirs used for conventional water supply. If the upper reservoir of a PSHEP is built on a hill (Dlouhé Stráně PSHEP) and the banks have an artificial lining, it is not necessary to make predictions about changes in the banks.

Processes affecting the shorelines of reservoirs in areas of diverse geology and climatic conditions where changes in water level are governed by an annual regime (Orava, Nechranice, and others) are not the same as those affecting a PSHEP because, in addition to the normal process of erosion, in this case an extra load is imposed by the regular and abrupt rise and fall of the water level. Consequently, dramatic changes in the level of groundwater in the banks will also occur leading to the following effects:

- Changes in stability of soil or rock due to sudden increases and decreases in the value of hydrostatic and hydrodynamic pressures; the effects of hydrodynamic pressure are different in soils and rocks and strongly influenced by permeability in particular;
- Changes in physical-mechanical properties of soils caused by the saturation of pores with water; consequently, the strength of soils and the strength on planes of discontinuity in solid rocks is decreased leading to large-scale destabilization of banks (rock falls, creep and landslips); and
- Washing out (elutriation) of the fine fraction from coarse detritus and gravel, etc., accompanied by a possible loss of slope stability.

Large daily fluctuations of the water level in reservoirs supplying a PSHEP have a specific effect on the pattern of erosion of banks and their surroundings. These changes are more intensive than those occurring in reservoirs with an annual or multiannual regime of water level fluctuation. The bank profile approaching the equilibrium state is usually formed more rapidly than in those dam reservoirs with a normal annual or multiannual regime of water levels.

9.2 Modification of Banks and Slope Failures

Priority should be given to assessing the overall stability of the area affected by a planned reservoir before construction begins. The susceptibility of the area to slope failures will be determined by making a thorough survey of the geological composition and structure in the area of the reservoir, using detailed field trips across both valley slopes. The effects of active failures and instabilities are usually clearly visible in the topography, but it is also necessary to assess the potential risk of deep failures on mountain slopes caused by long-term creep and by toppling and rock falls. The engineering-geological assessment of the overall stability of the area must be carried out in time to enable the plans for the layout of a hydro-engineering structure to be modified in accord with the geological recommendations that arise from this survey.

Representative geological profiles of certain sections of the shoreline with similar geological composition and structure should be compiled, as far as possible in the down-slope direction. The composition of the cover formations and the structure of the pre-Quaternary basement will be recorded in detail. The age, thickness, mode of deposition and the physical-mechanical properties of the cover formations must be described thoroughly. The proportions of the fine fractions and the characteristics of the framework of clasts in slope sediments should be carefully recorded. Deluvial formations in which silty clay, and/or debris with a high proportion of silty clay, rest on stony detritus with open cavities within the reach of the root system of the covering vegetation do not form stable reservoir banks. Banks formed on slopes of this type would be subject to rapid and extensive degradation due to the action of the water in the reservoir and the washing caused by surface precipitation. In the case of clastic sediments, it is important to determine whether the structure is matrix-supported or clast-supported and what the shape and size distribution of the clasts is so that it can be determined whether the framework of clasts will form a stable slope after the fine components of the sedimentary mantle have been washed out. It is important for the coarse fractions to be resistant to water action and not to become waterlogged. Where heavy stony debris accumulates at the base of a slope on top of loosely consolidated rocks, the situation is potentially unstable. When the underlying sediments become saturated by the reservoir, they will be weakened and predisposed to failure, allowing the overlying debris to slide down-slope into the reservoir. The mineralogy and texture of solid bedrocks must also be recorded carefully, together with measurements of their structural planes (bedding, jointing, cleavage and schistosity), their physical behaviour (friability, slaking, swelling capacity, shrinkage, stability and character of cements and infillings) and their state of weathering. Faults and other planes of weakness require special attention because they can act as surfaces for potential slope failures. All topographic features, such as dejection cones and debris flows, gullies, ravines and particularly slope failures must be mapped and described because they can be used to predict the behaviour of the slopes when the reservoir is brought into operation.

The bank zone follows the perimeter of the reservoir at the elevation of the water levels anticipated during the operating regime determined by the regulations for management of the reservoir. The width of the band covered by the detailed survey of the bank zone depends on the complexity of the local geological conditions, on the gradient of the slope and on certain other factors, such as interest in protecting the natural environment, and the safety of particular historical or archaeological sites of interest, etc. In areas susceptible to slope failures, the survey is usually carried out from the line of the future maximum reservoir level to a distance of at least 100 to 150 metres upslope. Profiles used for the engineering-geological survey of slopes should be sufficiently long to enable conditions governing the deposition of the cover sediments to be studied comprehensively, taking account of the evolution of the topography and the broader effects of cycles of erosion and deposition operating in the area. It is appropriate to make exploratory excavations (pits or boreholes) along at least one of the characteristic profiles in the area investigated. If the profile lies on a slope failure or in the area of a potential failure, it is appropriate to case exploratory boreholes to enable observations of slope movement. In these cases particularly, the basic pattern of geological profiles can be supplemented by a geophysical survey. An integral part of this detailed survey of vulnerable areas is an engineering-geological map, usually at a scale of 1:2 000 to 1:500. The investigation of slope movements and other potential slope failures at Ústie nad Priehradou in the Orava reservoir is an interesting example. Here, geophysical methods were used to monitor landslides on rotational shear planes (see Chap. 9.2.2).

When surveying the reservoir areas, it is necessary to take account of the parameters of the water reservoir (Tab. 9.2.1), and to pay particular attention to the classification of existing failures and predictions about their future development. Depending on the geological conditions in the area of interest and other relevant factors, the following types of failures may occur:

- Mass Flow: This takes place when soil, sand, clay and detritus are saturated by water and behave as an unstable fluid suspension with low viscosity flowing down-slope. Examples are solifluction that occurs when frozen ground thaws in the spring, and flows of saturated detritus or quick-clay on slopes saturated and destabilized by periods of high precipitation; and
- Landslides: These take place when masses of rock slide along discrete rotational shear planes, on combinations of shear planes, or on pre-existing planes of weakness.

A phenomenon transitional between flowing and sliding occurs when suffosion sliding is caused by the liquefaction slope failures or washout of sandy beds. Depending on the type of flow, slope failure can take the form of discrete flows or sheet run-off of surface layers.

Rotational failure on shear planes causes a definite break in the slope that is marked either by a cliff or a more extensive retrogressive landslide can form progressively by the gradual snagging of individual blocks against the slope. In this way, a composite

Table 9.2.1 Types of slope failures at dam reservoirs in the Czech and Slovak Republics

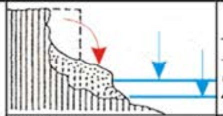
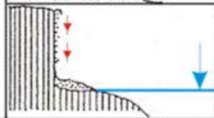
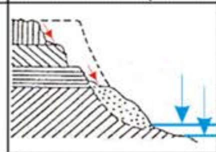
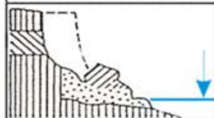
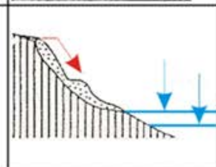
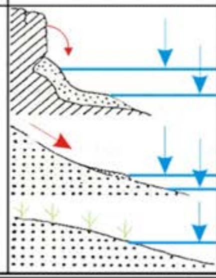
Type of bank change	Geological processes	Lithological composition	Height of abrasion wall [m]	Type of wall	Dam reservoir
Abrasion	Abrasion, weathering, falling-off, crumbling	Alluvial and deluvial soil, clayey sediment, shale, sandy clay, marlstone	0.3–10		Nechranice, Orava, Žermanice
					Nechranice, Olešná, Orava, Liptovská Mara
					Liptovská Mara, Orava, Nechranice, Hracholusky
Abrasion-landslide	Sliding, weathering, abrasion, suffosion	Clay, claystone, sandy clay, sandy shale, siltstone, silty clay, shale, marlstone	5–20		Liptovská Mara, Šance, Orava, Karolinka, Nechranice
					Orava, Nechranice
					Nechranice, Orava, Rozkoš
	Weathering, sliding, erosion abrasion, suffosion	Silty clay, sand, gravel, clay, shale	0.5–10		Jesenice, Orava
Abrasion -accumulation	Abrasion, accumulation, formation of peat bogs, biogenic effects	Clayey-sandy formation, marl	0.1–5		Liptovská Mara, Olešná, Orava

Fig. 9.2.1 The Vajont dam, left – a satellite view (www.Googleearth.com), right – a view of the landslide (a photo by P. Bláha - 2007)

shear plane is gradually formed, marked by an envelope formed by segments of separate subplanes. Landslides on composite rotational shear planes usually reach greater dimensions. For example, on the Orava reservoir at Ústie nad Priehradou the landslides are 250 metres wide and up to 350 metres long, including those parts underwater. The depth of the shear plane reaches 10–20 metres there. Slope failures triggered by shear along pre-existing planes of weakness can be far more serious. They can sometimes reach very large dimensions and have catastrophic consequences (e.g., Vajont, Figs. 2.3.31, 2.3.32, and 9.2.1).

The chief factors governing the pattern of slope movements on reservoir banks are structural, climatic, erosional and anthropogenic. The survey of slope failures in reservoir areas is usually complicated by the fact that a part of the mass is usually underwater and the exposed part of the slope failure is either in motion or in labile equilibrium. Survey work within the slope failure itself is usually difficult and mostly not practical. In most cases, only a part of a slope failure is visible so that it is not easy to judge either its extent or the conditions under which it formed on the basis of field investigations alone. The study of slope failures therefore requires a specific approach.

In the case of more extensive slope failures that pose an immediate threat to important facilities, a detailed planimetric survey at a scale of 1:500 combined with accurate levelling is recommended. Various types of sonar are used to survey slope failures below the water surface. This basic information about the morphology of the slope failure can be combined with geological mapping to provide an initial idea about the composition, extent, and causes of the landslides.

The investigation of the landslides around the Orava reservoir once again serves as an example of the procedures used. A preliminary idea about the development of the slide processes was obtained by study and interpretation of aerial images taken in 1953, 1955, 1962, 1971, and 1976 covering the period from just before the reservoir was filled up and after it had been filled. Drilling work in the landslides themselves was not practical for safety reasons, except for shallow holes drilled manually. In order to assess the migration of slide processes higher upslope, boreholes were collared above the zones depleted by the landslides. They were also used to make parametric measurements for a geophysical survey.

To monitor landslides on rotational shear planes and landslides triggered by shear on pre-existing planes, geophysical survey methods were used successfully for the first time at the Orava reservoir. The choice of suitable geophysical methods was based on the results of parametric measurements in boreholes to obtain a basic idea about the differences between the physical properties of slipped rocks and those of the rocks below the shear plane. The application of geophysics to the survey of landslides occurring along planes predisposed to shear was a simpler problem to solve.

9.2.1 The Area of Landslides at Goral

Several landslides were formed below the Goral chalet as a result of the erosion of the banks of the reservoir and due to other causes. These were an immediate threat to the recreational facilities offered by the hotel. The upper boundary of the zone of depletion had already reached to less than 12 metres from the foundations.

A review of previous survey work combined with new field observations showed that Neogene rocks cropped out below the threatened hotel and the contact between the Neogene and Palaeogene rocks was located higher on the slope (Fig. 9.2.2, lower left). It was concluded that the outcrop of Neogene sediments was a remnant of those deposited on the eroded Palaeogene basement in a bay of the Neogene sea. The unconformity had a dip of 15–20° towards the reservoir. The investigation showed that the Neogene sediments were sliding on the Palaeogene erosion surface and that the process would be

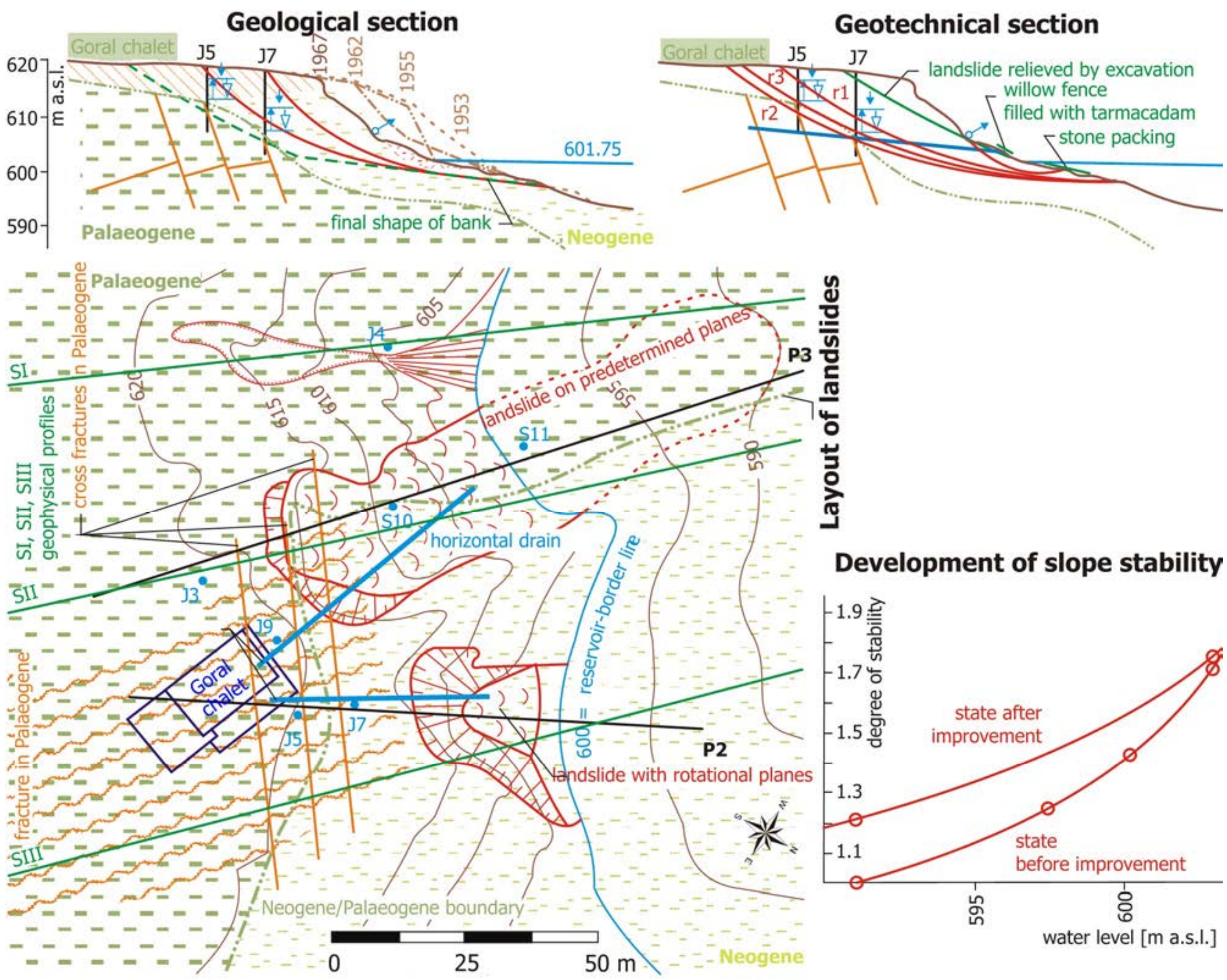


Fig. 9.2.2 Goral landslide

complete after all the Neogene beds had slipped down. The survey was therefore focused on defining the boundary between the Palaeogene and the Neogene.

For the survey, both engineering-geological procedures (study of topography, geological composition and structure, detailed surveys of landslides and laying out of observation points and straight lines, subsurface exploration work) and geophysical methods (resistivity sounding and profiling, seismic sounding) were used. The survey revealed that the whole facility threatened by the landslide was already standing on Palaeogene rocks. However, in the area of the chalet these were deeply weathered and even altered into clayey sand with geotechnical properties similar to those of the Neogene sediments. The deep weathering was located along a tectonic zone about 50 metres wide, the position of which was traced successfully by resistivity profiling. The situation was thus quite complicated because only part of the area of the slide could be explained in terms of sliding on a pre-existing plane, i.e. by sliding of the Neogene over the Palaeogene unconformity. In the area of the tectonic zone, landslides in the Palaeogene also occurred on rotational shear planes due to the similarity between the physical-mechanical properties of the Neogene sediments and the weathered tectonically disturbed Palaeogene sequence.

It is clear from the picture of the landslide on rotational shear planes in the area of the tectonic zone (Fig. 9.2.2, upper left) that the hotel and its recreational facilities (Fig. 9.2.3) would be ruined unless remedial measures were undertaken (drainage of the landslide area by horizontal drains, reinforcement of banks to provide protection from erosion, etc.).

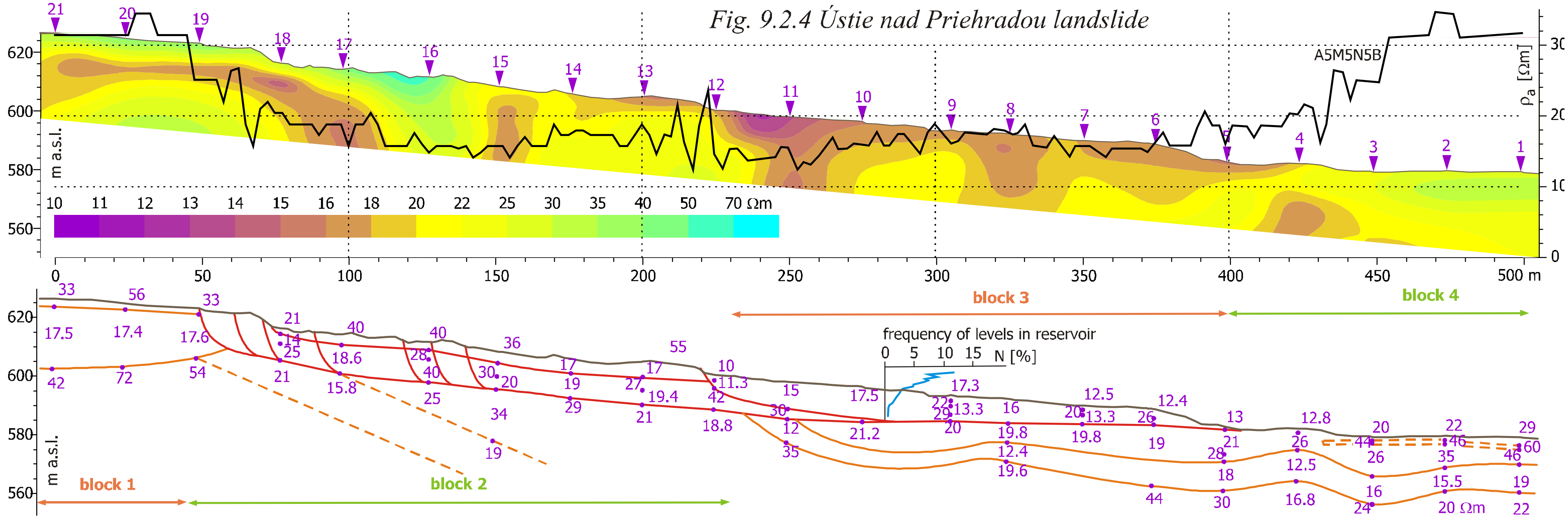
Fig. 9.2.3 Goral landslide (a photo by O. Horský - 1969)

In 1968, the landslide was in a state of temporary equilibrium with a degree of stability m = 1. By substituting the values for upward pressure, m=1 and cohesion c=0 in a stability equation, the angle of internal friction on the shear plane was determined to be $\varphi = 18^{o}30'$. By substituting these values back into the calculation, the conditions of stability for the full reservoir and for sudden drops in the height of the water level to different elevations were determined. The same calculation was made for different shear planes allowing for the effects of remedial measures. A degree of stability m = 1.3 was determined to be safe; under these conditions the landslide would not be activated even if a sudden drop of the water level by 4.5 metres took place. Following remedial measures that included covering the range of the most frequent fluctuations in the water level with stone packs, placing a willow fence within the range of the breaking waves, and the partial redistribution of materials and reducing the inrushes of water by stone packs, the degree of the slope stability was increased to a value of 1.2, even in circumstances in which the level of the water dropped suddenly to an elevation of 591 m a.s.l. which is the level at which the shear plane of the landslide crops out on the shore (graph in Fig. 9.2.2). After two horizontal drains had been installed in the landslide, the degree of stability rose to a value of 1.4 at the same ground elevation. These remedial measures were carried out in 1969–1970 and the area has remained stable ever since.

9.2.2 The Landslides at Ústie nad Priehradou

In the bank cut by the River Čierna Orava at Ústie nad Priehradou, landslides in the Neogene sediments had already occurred before the reservoir was filled up. After submergence, the accumulated material was washed out and removed by erosion. Consequently, the landslide process was reactivated. At present, landslides have reached 180 metres back into the slope. A field path has been destroyed and a cemetery and a church are now under threat. Not counting the part below water, the landslide below the church covers an area of 80 × 150 metres, and the landslide below the path covers an area of 250 × 300 metres. It was too dangerous to carry out subsurface exploration work within the active landslides. The core from a borehole over the zone of depletion showed that the shear plane was located at a depth of 12–14 metres below surface. This depth was compatible with the topography and the position of the shear plane in a longitudinal section through the landslide suggested that the landslide formed progressively on rotational shear planes. The correctness of these deductions was fully confirmed by a geophysical survey. The position of the rotational shear planes was determined using a combination of resistivity sounding and resistivity profiling. Parametric measurements in a borehole behind the crown of the landslide showed that the Quaternary sediments have relatively higher resistivities (70–100 Ωm); the Neogene clay has low resistivity (17–32 Ωm). Variations in the resistivity of the Neogene clay are caused by intercalations of sand and lignite that increase the apparent resistivity to about 30 ohm-metres as compared to the “pure” clay, which has a resistivity of 10–20 Ωm. An illustration showing how the shear plane was located at a depth of 12–14 metres by the method of resistivity sounding and profiling is given in Figure 9.2.4. The surface of rupture obviously follows a zone of cracking in the clay

Fig. 9.2.4 Ústie nad Priehradou landslide



that is clearly visible in the results of resistivity profiling. The shape and course of the shear plane is reflected by the contours of apparent resistivity. The individual snags on shear planes are indicated by distinct as well as less distinct minima in the contours of apparent resistivity.

Significant pressures are developed in the groundwater within the Neogene sediments at Ústie nad Priehradou due to the variable permeability of the rock environment. In layers dipping towards the reservoir, water flows into sandy beds and seams of lignite. Mostly, these beds are not stratigraphically continuous, and lenticular pockets of saturated sediments dipping towards the reservoir and capped by a less permeable roof of clay have formed. When the level of the water in the reservoir suddenly drops, water cannot drain out of previously saturated parts of the slope quickly and the impermeable cappings of clay can be burst open by the excess pressure of the water trapped in the lenses of permeable sediment. In the winter season, when the water level in the reservoir is regularly low, this capping layer usually freezes. If there is a sudden thaw, the trapped water breaks out and the capping is washed away due to the pressure of escaping groundwater. Under unfavourable topographic conditions and in combination with other processes of erosion, the stability of the slope is disturbed and landslides such as those at Ústie nad Priehradou are formed.

The flow of groundwater through more permeable beds causes an extensive underground suffosion of the sediments. Above the high cliffs formed by erosion at Ústie nad Priehradou, at a distance of 10 to 30 metres from the face, a row of pseudo-sinkholes, now more than two metres deep, have formed in regular lines. These fill up with water during wet weather. The infiltrated water flows out at the base of the cliff and during periods when the water level in the reservoir drops it flows out even lower on the exposed slope. The outflows of water become turbid during periods of heavy precipitation, providing clear evidence of the ongoing underground suffosion of sediments and thus also of the degradation of the affected soils, leading to a significant reduction in their strength parameters. Sometimes, outflows of water are so vigorous that marked depressions are formed. Lines of sinkholes create potential scarps for the gradual extension of landslides and collapses higher on the slope. The process of suffosion is prevalent particularly during the thaw of snow and ice in the spring when the water level is often lowered by more than 10 metres and the thawing slopes are saturated by water.

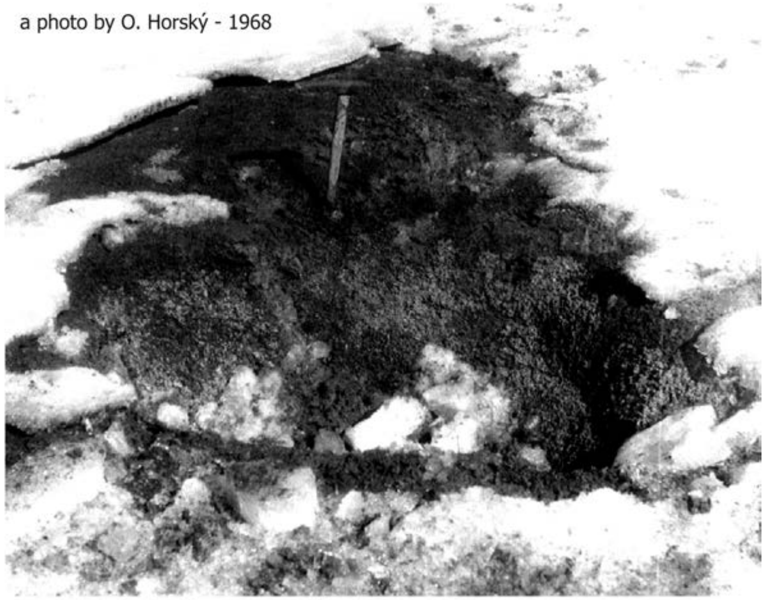


Fig. 9.2.5 Suffosion sinkhole

An illustration of a suffosion sinkhole from the Ústie nad Priehradou site is given in Figure 9.2.5. Other illustrations of the changes caused by suffosion in the banks of reservoirs are given in Chapter 9.3. These illustrations also show that the holes through which water flows out are always situated just above the water level in the dam lake.

Relatively low temperatures in the winter season lead to deep freezing of the slopes. Water in pores and fractures in the rock mass increases in volume, causing the expansion and cracking of the marginal parts of cliffs adjacent to slopes exposed above the water surface. This causes a change in the physical-mechanical properties of clayey rocks and increases their washability (susceptibility to scouring). When the frozen surface layers thaw, the saturated and expanded mass becomes slushy and susceptible to collapse and flow and falling blocks of ice also play a part in eroding the banks. In contrast, during the summer season, the banks dry out in the sunshine and shrinkage cracks are formed, accompanied by crumbling and falling away of material from the cliffs and banks. At the foot of cliffs distinct cones of talus are formed. These are quickly eroded by the breaking waves and removed (Fig. 9.2.6). Under intense pounding by waves, water easily penetrates into shrinkage cracks and the cliffs are further eroded. Similar effects are caused by intensive rainfall.

Fig. 9.2.6 Crumbling and collapse of abrasion walls (a photo by O. Horský - 1968)

9.2.3 The Landslides at Osada Site

At the Osada site, a section of the bank about 1.5 km long has been affected by bank degradation. The banks consist of Neogene calcareous clay and claystone, locally with beds of poorly consolidated sandstone. The Quaternary cover consists of layers of rewashed loess and sandy glaciofluvial gravel from three to five metres thick. Before the reservoir was filled up, the sequence had been deeply eroded by the River Čierna Orava. On the left bank cut by the river, steep slopes with a gradient of 24 o and up to 40 metres high had been formed. After the reservoir was filled up, they were intensively eroded and, as a result, fossil landslides were reactivated which led, to new slope failures in the banks. The stability of the entire section of bank was further disturbed by the marked fluctuation of the water level in the reservoir. Landslides were progressively formed on rotational shear planes, migrating retrogressively into the slope (Figs. 9.2.7 and 9.2.8).

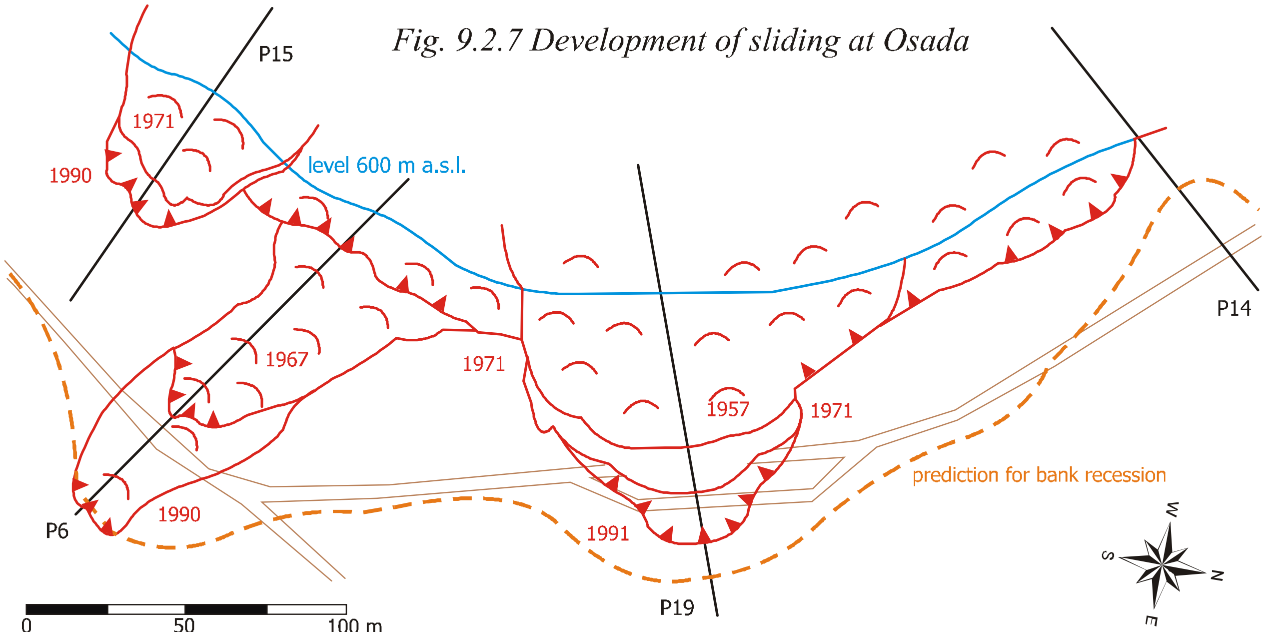


Fig. 9.2.7 Development of sliding at Osada

The gradients of the abrasion platforms on the Neogene sediments at Osada are gradually stabilizing with an average slope of 6.5^{o}, but they are still being cut downwards and recede with the shoreline, which is determined partly by the regime of fluctuation in water level during the year, and partly by the very low

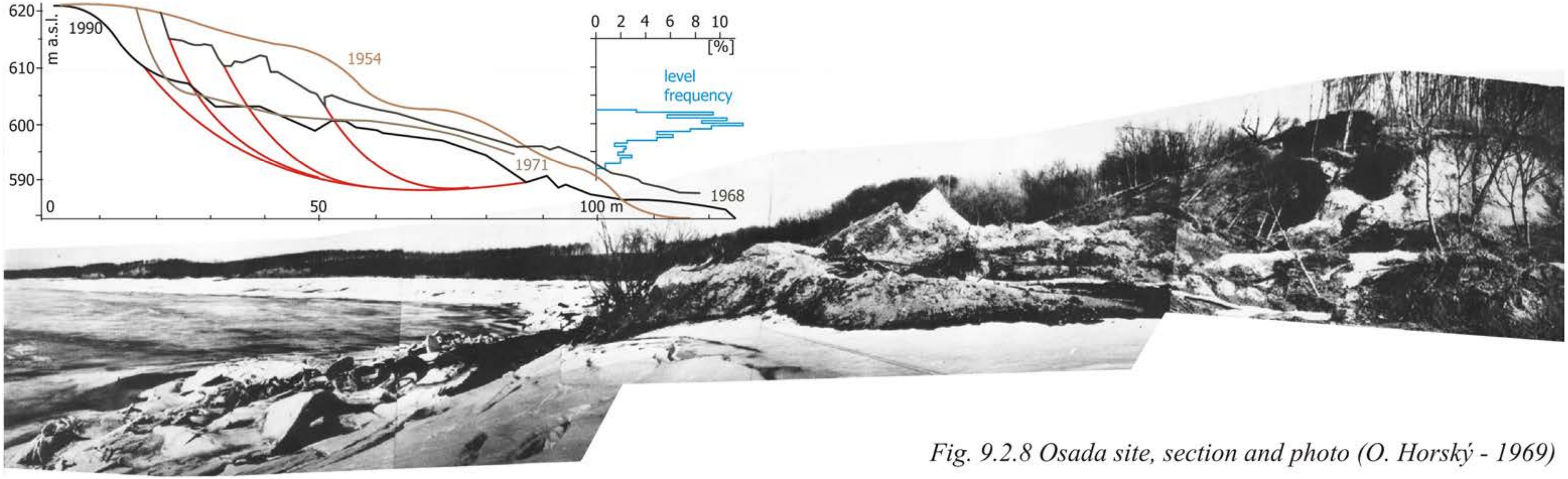


Fig. 9.2.8 Osada site, section and photo (O. Horský - 1969)

resistance of the Neogene sediments to erosion. Their strength, and thus resistance to erosion, was reduced by unloading as the overlying beds were eroded and also by weathering. Geophysical measurements showed that weathering reaches a depth of 18 to 20 metres below the surface. However, the decisive factor responsible for the reduction in resistance to erosion is the fracturing that took place during the process of sliding.

Fig. 9.2.9 Erosion furrows

Other factors also led to a decrease in the stability of the banks and subsequently to sliding. In the high cliffs created by erosion of outcrops of sandy beds, outbursts of groundwater are observed. When the face of the cliff freezes in the winter season, the relatively permeable layer becomes saturated and the pressure builds up behind the face. As a result, during a sudden thaw, the wall breaks out and partly collapses. In combination, the rapid thaw in the spring months and heavy rainfall in the summer and autumn months have a markedly adverse effect on the stability of the cliffs and banks. They cause extensive erosion of exposed slopes and the loosened sediments accumulate below landslides and cliffs. In some cases this leads to the activation of fossil landslides and to the formation of new landslides (Fig. 9.2.9). Because the cliffs are exposed to rapid variations of temperature over a wide range, very intensive weathering takes place. Air temperatures fall to -30 °C in February, whereas in the summer months they sometimes rise to 25 °C or more. The disintegrated sediments accumulate below cliffs, both on the beach platform and below the water level and gradually settle in calm bays and deeper parts of the reservoir. Erosion of the accumulations below cliffs and on the slopes of the beach platform is particularly intensive in the spring months when the water level in the reservoir falls (Figs. 9.2.10 and 9.1.2).

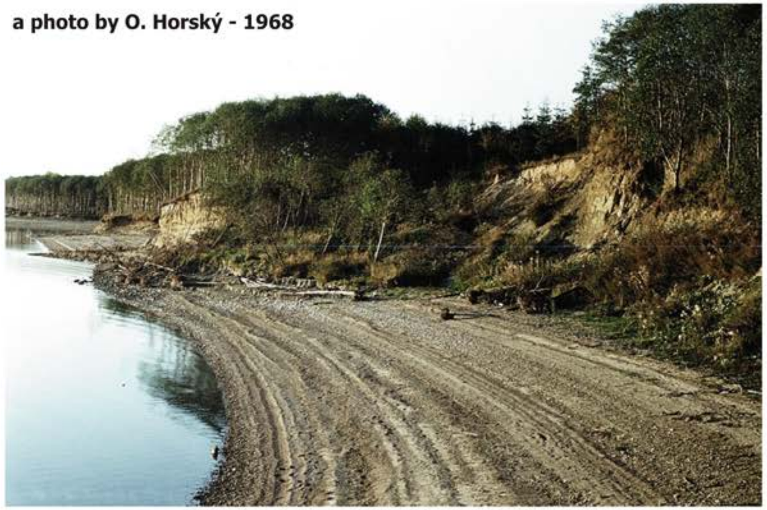


Fig. 9.2.10 Abrasion cliffs

Fig. 9.2.11 Gouging of bank by sliding ice block

Mass wasting in the form of talus and rock falls also occurs at the Osada site. The effect of ice on the formation of banks is typical of the whole Orava reservoir; but the action of ice at the Osada site is particularly intensive (Fig. 9.2.11). As at Ústie nad Priehradou, the effects of suffosion at the Osada site are also widespread, mainly in re-washed loess and in the eluvial deposits resting on the Neogene basement. Masses of slipped material are being deposited below the collapsed slopes and extend out into the reservoir below the water surface. Hence, the initial equilibrium of the abrasion platforms is repeatedly disturbed and the entire shoreline affected by the process of sliding migrates towards the reservoir. In the slipped materials, the cycle of erosion again occurs, gradually leading to the stabilization of the abrasion platforms. For the reasons given, the prediction of changes in banks of the erosion-landslide type is different from that of the abrasion type (Fig. 9.1.13).

9.3 Dams for Special Use

Not all dams serve as conventional hydro-engineering structures for supply of water whatever the use. Dams with special use certainly include dams of various sludge settling basins. After having constructed the basic dam, this structure is further raised using now the material of the sludge settling basin. By suitable placing waste piping and by depositing water-laid material, the dam of the sludge settling basin is gradually raised so that the required volume can be achieved for depositing waste material. Figure 9.3.1 shows the dam of the sludge settling basin Erdenet in Mongolia, which serves for depositing pulp at one of the largest world copper deposits. The problems with such structures are not small in any case and today the design of such dams and related


Fig. 9.3.1 Dam of Erdenet deposit

engineering-geological surveys and monitoring represent a separate scientific branch. It is necessary in dams of such a type to constantly monitor seepage and dam stability. Failure to carry out such work or omitting its correct regime can result in such tragedies, as happened at Kolontár, Hungary, in a reservoir of wastes generated from the production of aluminium. An example of a survey for seepage that occurred in the dam at Dolní Líštná at the sludge settling basin of an ironworks company, using geophysical methods, is described in Chapter 6 (Fig. 6.1.6).

Another example is a dam designed for the protection of a landscape lying below such a dam. Such structures are constructed for capturing extra heavy torrential rains or for catching a specific type of slope failures, i.e. for eliminating dangerous effects of debris flows. These slope failures of the flow type reach the velocity of movement of the sliding material – up to tens of kilometres per hour, or even more. The moving material of this type of slope failures has the character of a heavy liquid rather than soil. Figures 9.3.2 to 9.3.4 show three possible types of dams used for these purposes in Uzbekistan.

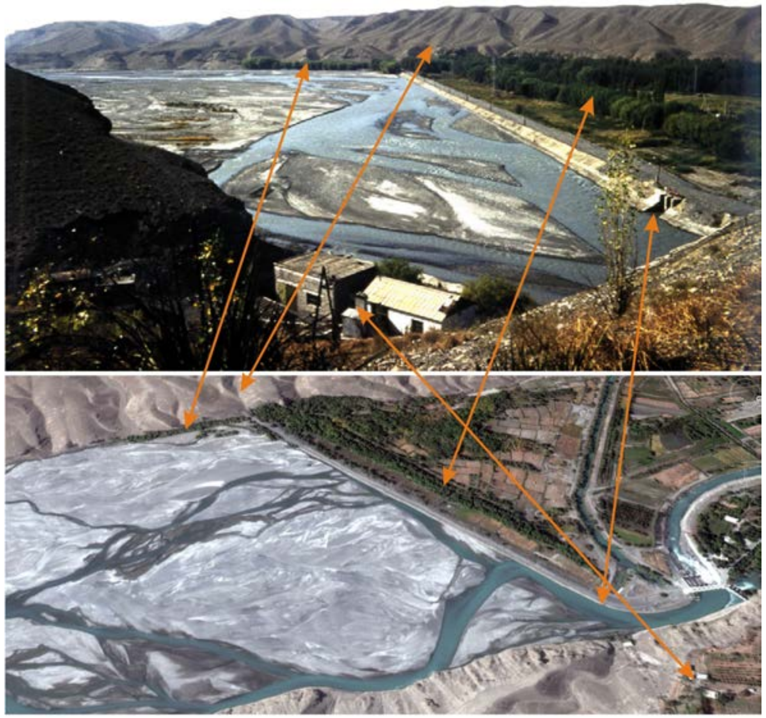


Fig. 9.3.2 Protective dam on the River Sokh (a photo by P. Bláha 1984 + GoogleEarth)

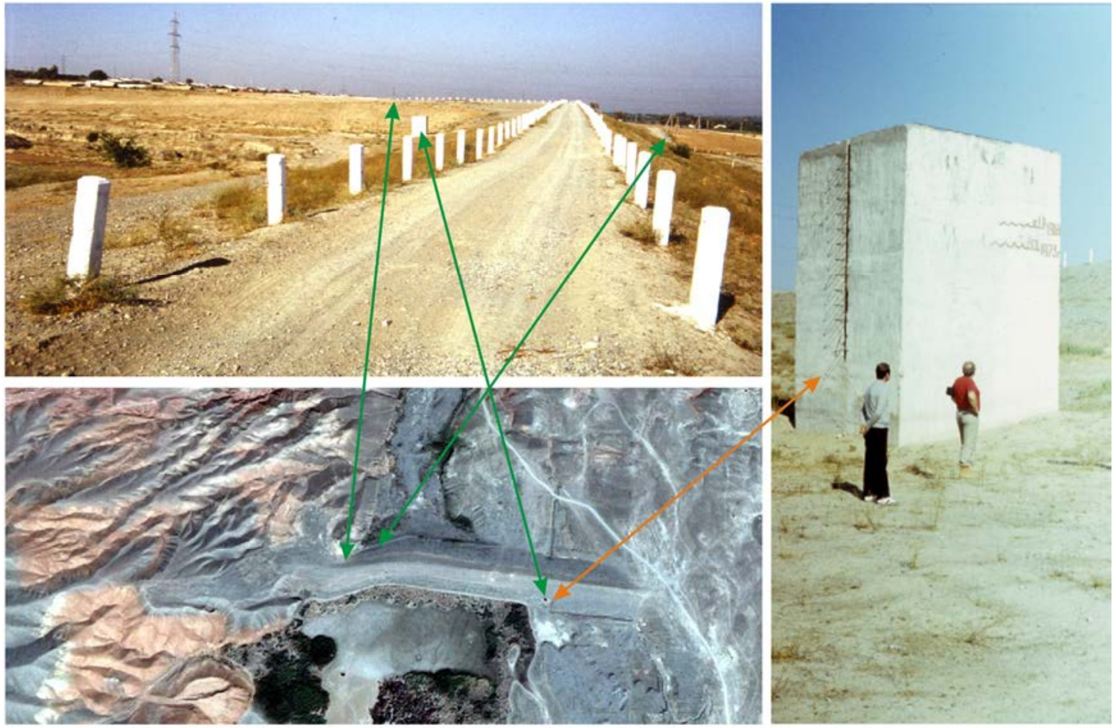


Fig. 9.3.3 Protective dam on the River Okhchi (a photo by P. Bláha 1984 + GoogleEarth)

The protective dam on the Uzbek River Sokh in the Fergana Valley (Fig. 9.3.2) has the task to protect the towns of Otchi and Kokand in its lower course from floods. The upper part of the figure shows a photograph of the dam from September 1984, the lower part a photograph from the server GoogleEarth from 8 October 2011. Thin lines on the figures connect identical places in both photographs. The relatively low dam 780 metres long is built on a typically braided stream and its upstream side is covered with a concrete diaphragm. It serves both as a sealing element and as a protection against damage to the body itself of the embankment dam by destructive effects of abrupt floods.

Behind the turn of the dam (the right-hand side of the lower figure), there is a canal conveying water to a smaller hydroelectric power plant. A small spillway provides a safe discharge of water masses farther downstream when the flow through the power plant is closed. Sediment deposits, chiefly gravel or sand, are used in dry seasons for extraction of aggregate to be added to concrete or for other construction purposes as well.

The protective dam of the second type (Fig. 9.3.3) constructed across the valley of the River Okhchi (Kyrgyzstan) above the town of Rishtan (Uzbekistan) protects it from debris flows. Again, the figure shows the situation from 1984 (the upper part) and the situation from 2007 (the lower part). As in the case of the preceding figure, the same places are connected with thin lines. The safety object is situated in the lower course of the river. The protective dam is constructed in a wide dry valley, or in a valley of an intermittent stream (wadi). The relatively long and low dam provides a sufficient flood-control storage capacity for capturing debris flows and flash floods. The crest of the dam also serves as a local road. A special outlet structure (Fig. 9.3.3 right) serves for release of water. It releases water only through a narrow slot that does not allow water masses to carry coarse gravel or loosened parts of the rock mass farther downstream. In certain cases, the outlet slot of the safety object is usually sited on that side of the object which faces the dam body itself.

The last type of protective dams is in Fig. 9.3.4. This type of dams is amply used in the Fergana Valley and has the character of an all-concrete body that is built across the upper courses of rivers. Water outflow is not accomplished by special spillway structures, but free spaces – "safety windows" – are left in the body itself of the concrete dam during its construction. These mostly concern small structures that are built in cascades across the river. With the tremendous carrying capability of rivers in the upper courses, it is natural that the flood-control storage of such dam reservoirs is filled up relatively soon. Even in this case, however, the protective purpose of such a system remains preserved. The longitudinal profile of the stream is strongly changed by filled "dam reservoirs". Water masses flow in sections of a relatively smaller head and overcome the difference in heights in the form of small waterfalls. One of the authors of this publication saw these dams after five years of operation, fully filled by coarse-grained material, but still successfully serving for the protection of the lower course of the river.

Fig. 9.3.4 Protective dam for small rivers (a photo by P. Bláha 1984)

Similar effects can also be achieved by far more "primitive" methods, e.g. also by simple wooden barriers (Fig. 9.3.5). Any reduction in the velocity of the water stream will decrease its carrying capability and change the longitudinal profile of the stream. As proved by a preliminary study in Slovakia in the Kysuca municipality of Lodno, where a system of "dam reservoirs" was built across small tributaries of the River Kysuca; in the first months of their life they held back about 760 m^3 of mud and sediments, which would otherwise have been washed away to the river during the season of higher water stages and subsequently to the Hríčov dam reservoir. The authors of the study found out that the mud behind the dams had contained up to 41% of water on average, which during the dry season gradually flowed out from the mud, and after rains the mud again saturated by water. The mud contained 310 m^3 of water on average, which permanently flowed out, and after rains the mud was again saturated, thus permanently improving the discharge regime of brooks. A system of floodplains has gradually been formed and the water-resources balance has improved not only in brooks, but also in the surrounding environment in which the yield of local water sources has increased. Naturally, such designs are suitable in inhabited valleys with a relatively high density of habitation.

Fig. 9.3.5 Protective barrier against erosion

We want to describe a complex system of the protection of a large city from the danger of abrupt floods, debris flows and glacial lake outbursts, giving an example of the City of Almaty, which was founded in 1854. Almaty (the former capital of Kazakhstan) lies at altitudes from 550 to 1,650 m a.s.l. The major part of the city is found on central Asian steppes at an altitude of just below 600 m a.s.l. The city lies closely to the foothills of the Alatau Mountain Range, with the highest summits reaching over 4,000 m a.s.l. Some parts of the city are built on the dejection cones of the Rivers Aksay, Bolshaya (Greater) Almatinka, Malaya (Lesser) Almatinka and Talgar.

Steeply descending valleys of these rivers are very sensitive to the formation of debris flows or mud flows. Since the city foundation, five disastrous debris flows have been recorded in the valleys of the Rivers Malaya Almatinka and Bolshaya Almatinka. In 1887 and 1910, debris flows were activated by earthquakes; in 1921, by heavy rains; and in 1973 and 1977, by breaching moraine-dammed lakes high in the mountains. The most serious of them was a debris flow in 1921 on the Malaya Almatinka. At that time, 500 people died (the City of Almaty had then 45,000 inhabitants). The total volume of the debris flow was 10 Mm^3, of which were three million m^3 of rock. The maximum discharge rate of the debris flow in the mountains was 5,000 m^3/s, below the mountains 500 m^3/s. If the debris flows brought up to 500 m^3/s of material to the city, this would mean that all the material would be carried to the city by the debris flow in a little longer than five hours. It is estimated that in the case of a recurrence of such a debris flow early in the 21st century, the incurred damages would exceed on the order of hundreds of millions of US dollars.

To protect the city from the danger of debris flows, six dams (Fig. 9.3.6) have been constructed across individual streams. The centre of Figure 9.3.6 shows the situation of protective dams above the city of Almaty as seen on GoogleEarth. Along the edges of these central maps, there are views of the individual protective dams, with the names of the authors of the photographs, as the individual photographs are authorized on Google Earth.

Dam 1 across the River Aksay is about 160 m long and about 15 m high (estimated according to the display on GoogleEarth). Photograph 1A shows water-laid sediment composed chiefly of coarse gravel. The body itself of the relatively simple dam is visible in the centre of the photograph.

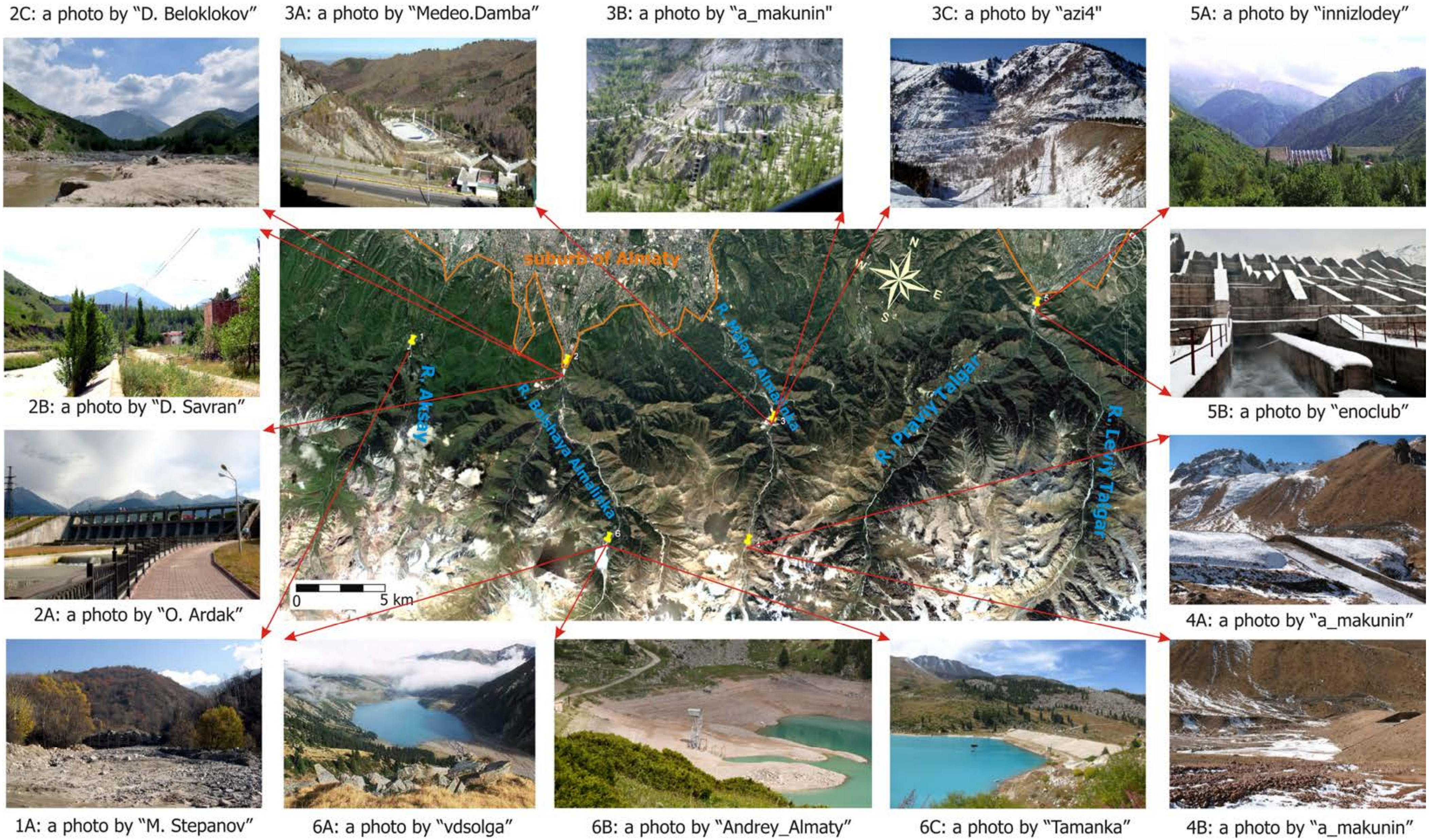


Fig. 9.3.6 Protection of the City of Almaty from debris flows

The second dam stands across the Bolshaya Almatinka and is 422 long (Yesenov and Degovets, 1982). The height of the second dam is 40 metres and the dam construction was completed in 1982 and the created flood-control storage had a volume of 14.5 Mm^3.

Photograph 2A shows the spillway structure of the dam; Photograph 2B shows a general view of the downstream side of the dam, now already covered by a new forest growth. Photograph 2C shows the character of sediments in the flood-control storage. In this case, coarse- as well as fine-grained sediments occur, which are extracted in the summer season as construction material. The sixth dam 360 m long and about 35 m high lies at an altitude of 2,520 metres. The original dam of the glacial lake was raised by ten metres and water is diverted from the lake so that the water surface cannot exceed the critical limit. Photograph 6A shows a general view of the lake; Photograph 6B shows a half-empty reservoir with an intake structure; and the final photograph 6C shows the level of the water surface during the period of spring thaw.

The largest protective dam Medeo (3) was constructed across the Malaya Almatinka just above the sports complex Medeo (Fig. 9.3.6. – 3A). The winter resort serves not only as a place for various championships, but also as a high-mountain training centre. The fundamental part of the dam now 530 metres long and 150 metres high was constructed in the 1960s, using a series of four explosions with a total amount of 1,800 t of explosive material and completed with an explosion of 3,600 tonnes of ammonium nitrate. On 4 April 1967, the dam was reinforced with material obtained by an explosion of 3,900 tonnes of ammonium nitrate as well. This dam was 110 m high (Yesenov and Degovets, 1982) and created a volume of 6.2 million m^3 of the sedimentation area. It turned out that the dam had been finished just in time because another disastrous debris flow occurred on 15 July 1973. The dam successfully stopped the debris flow of volume of 5.5 million m^3, of which 3.8 million m^3 being solid material that deposited in the reservoir behind the dam, hence its bottom rose to an elevation of 1,835 metres a.s.l. The debris flow up to 20 m high with a discharge rate of 5,200 m^3/s carried blocks of rock weighing up to 300 tonnes at a velocity of up to 40 km/h. The debris flow in the channel of the Malaya Almatinka destroyed steel barriers and gabion walls about 10 metres high. Two thirds of the volume of the flood-control storage of the 110-m-high dam were filled up. To increase the active storage of the reservoir, it was decided to finish the dam to the final height 150 m using the conventional technique, which was carried out in 1975-1980. This increased the flood-control storage of the reservoir to 12.6 million m^3. Photograph 3B shows a complicated intake structure on the left bank of the valley. Water is transferred from the flood-control storage to below the dam through two tunnels 540 and 460 m long, respectively, and with a profile of 17 m^2. The tunnels on the downstream side of the dam pass on to a reinforced-concrete tunnel. The intake structure is formed by two shaft collectors and one tower structure. Photograph 3C shows a view of the “upstream” side of the completed dam. The fourth dam across the same river at an altitude of 3,100 metres is 290 metres long and 17 m high. This dam is still unfinished and is not planned to be completed in the foreseeable future. The suspension of its construction is explained by the success of the dam Medeo and by a lack of financial resources. Photographs 4A and 4B show views of the dam from its upstream and downstream sides, respectively. A spillway structure was planned to be placed in the centre of the dam.

In addition to these large dams, other protective measures were also taken on the Bolshaya Almatinka and on the Malaya Almatinka. These were undertaken to reinforce river banks, to construct small, mostly gabion dams which create local settling basins. Besides these measures, a monitoring system is also installed along the rivers, enabling early warning and adoption of necessary measures when dangerous situations emerge.

The last protective dam (5) lies below the confluence of the Rivers Pravyi (Right) Talgar and Levyi (Left) Talgar. The dam is 360 metres long and about 40 metres high (estimated). The centre of the dam (Fig. 9.3.6 - 5A) shows a visible spillway structure that has a robust system constructed on the downstream side for breaking water masses flowing over the spillway edge during high water stages.

9.4 Silting of Water Reservoirs

Deposition of sediments brought by a water stream into a dam reservoir is a phenomenon that has been bound to construction of water reservoirs since the beginning of their construction. This problem was tackled by the Czech builders Štěpánek Netolický and Jakub Krčín of Jelčany as early as the Middle Ages. When constructing systems of South Bohemian fishponds, they also built bypass channels that diverted water outside the fishponds in the event of floods, and hence in the event when water carries a large amount of sediment load. The fact that both the fishponds and the bypass channels have served their purpose to day also shows that they understood their craft very well.

By damming up a river and creating an artificial water reservoir, a deep change takes place in the hydrological and limnological regime of the river system concerned. Particularly projects of large dams cause irreversible ecological changes both near the shoreline of the water reservoir, and in the areas above and below the dam. Dramatic transformations take place in the size of the river discharges below the dam, which will strongly change after the water reservoir has been commissioned: the quality, amount and use of water will change. The changed system of discharges strongly affects the character and level of groundwater table and the quality of groundwater; the aquatic biosphere and the surrounding phytosphere is gradually affected, which also leads to the influence of the living conditions of the population. The character of the living creatures will be changed in the river, in the reservoir as well as in their surroundings. At the same time, the character of sedimentation in the river basin will be strongly changed.

Fig. 9.4.1 Sedimentation and erosion on the Orava reservoir (a photo by O. Horsky - 1992)

By constructing an artificial reservoir, the natural curve of the longitudinal profile of the river will change, causing a change in the conditions, by which the erosion and accumulation processes along the stream of the river were governed so far. In the area of the water reservoir, a new lower erosion base will form; in its upper part, deltaic deposition of sediment load will begin to take place and then their erosion will occur when the water level in the reservoir is lowered (Orava reservoir – Fig. 9.4.1, Akhangaran reservoir – Fig. 9.4.2). In addition, the hydrological and limnological conditions of the river system will strongly change, causing a change in the flow, amount and use of water, a change in its quality and in the occurrence of biotic creatures, and finally also a change in the mode and amount of sedimentation both in the reservoir and in the river basin. Also, the character of the erosion and accumulation processes below the dam will change. The silting itself of the reservoir depends on its size and shape, on the velocity and size of flow in the reservoir, on the local geological conditions and, last but not least, also on the development of geodynamic processes in the reservoir area, such as landslides, abrasive and erosive phenomena, debris cones on the slopes (Akhangaran reservoir, Fig. 9.4.3), and the like.

Fig. 9.4.2 Sedimentation on the Akhangaran reservoir (a photo by P. Blaha - 1984)

The amount of sediment load carried into a reservoir is variable, dependent not only on the local geological and topographic conditions, but also on the climate, particularly on the amount of precipitation and its distribution during the year, on the gradient of the river stream, on the angle of slopes and their vegetative cover and on the method of agricultural land use below the dam. It holds that the largest amount of sediment is produced during extreme discharges on the given river and is the fastest in the areas of arid climate during extreme precipitation. There are known cases when a water reservoir was silted up during its twenty-year-long operation.

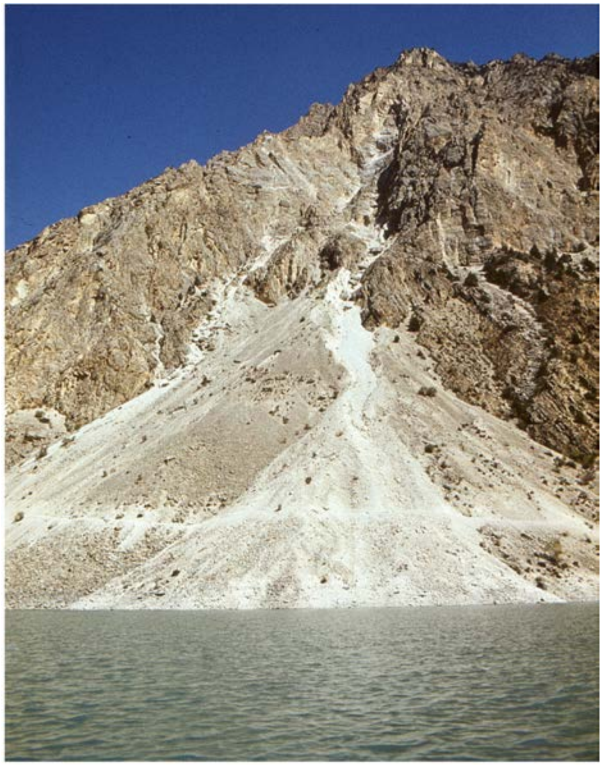


Fig. 9.4.3 Dejection cone on the Akhangaran reservoir (a photo by P. Blaha - 1984)

A specific case of silting was encountered in the Philippines where two dams have been constructed, one just behind the other, on the River Agno. The upper “Ambuklao” dam, practically 129 metres high, was completed in 1956. Some 8.3 kilometres downstream, the Binga dam with a height of 107 metres was also built. Over the time of its operation, the Ambuklao reservoir was more or less filled up by alluvial sediments (Fig. 9.4.4). The amount of sediment in the reservoir eventually reached a level that made it impossible to use the reservoir for energy generation. In 2007, both the reservoirs were privatized under single ownership. The new owner decided not to restore the Ambuklao reservoir to full operation, but to use the reservoir only as a settling pond to protect the Binga dam downstream on the River Agno. The plan is to use the power plant with the originally installed capacity of 75 MW under a constant load and to sustain the function of the reservoir only to the minimum extent required. The proposal made by a Czech company for a more fundamental clean-up of the area surrounding the intake structure that would restore it to full operation has not been accepted.

Sediment load carried by river streams is in the form of fine clayey and silty particles, forming a suspension, and in the form of coarse sediments, particularly sand and gravel; however, in the period of catastrophic floods, even debris or blocks of rock begin to move. During larger floods in some reservoirs, sediment load in the form of fine suspension does not settle in them and only flow through them. The Old Aswan Dam on the River Nile in Egypt can be served as an example:

its gates were opened yearly to pass beneficial muddy sediment load rich in nutrients so much needed for conventional farming in the near-shore areas of the river. But after commissioning the Aswan High Dam in 1971, everything has changed because the velocity of the flowing water has decreased so much even during floods that it also enables sedimentation of suspension load. Egypt has gained so much needed electric energy, but agriculture has become dependent on artificial fertilizers.

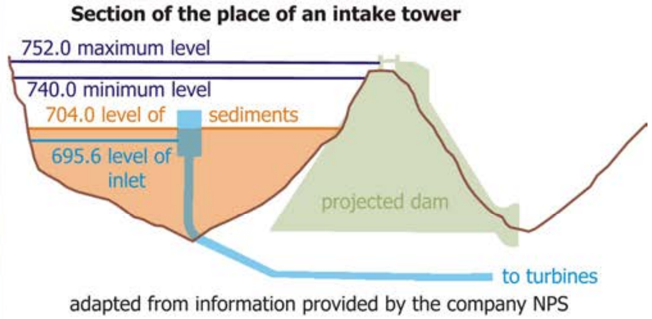


Fig. 9.4.4 Functional dam and non-functional powerhouse (a photo by J. Knotek - 2009)

Fine sediment load in large water reservoirs is settled on their bottom in the form of fine muddy sludge, or moves in relationship to the velocity of flow at the bottom of the reservoir, or at the base of slopes in the areas of the former river bed in the form of muddy flows. At a large drawdown of the water surface for operational or other reasons (for example, when creating the flood-control storage during the expected spring thaw of snow cover in the river basin), then they are eroded and slide down (see Fig. 9.1.2).

An interesting case of the monitoring of silting of the Bakaru water reservoir in Indonesia is given by Hadi Susilo and Ismu Nugroho in 2008. This small reservoir was designed to capture six-hour precipitation and to enable the operation of two, at the next stage four, turbines, each with an output of 63 MW. The total volume of the dam lake was designed at about 7 Mm^3. The required volume for the operation of four turbines was 2 Mm^3 and the assumed annual volume of sediments was 0.13 Mm^3. The real pattern of reservoir silting is depicted in Figure 9.4.5. The graph shows that sedimentation was much higher than expected and that the dam lake was gradually practically all silted up. The remaining volume of water, 0.8 Mm^3, is hardly sufficient for the operation of two generators commissioned at the first stage of water reservoir construction. Three attempts to excavate the sediments were not successful. During the first two attempts to clean the reservoir, the excavated volume of sediments was roughly by an order of magnitude lower than the planned. In the third attempt, the volume of excavated soil was increased to 60 % relative to the planned amount. Attempts to release the accumulated sediments were more successful than their excavation. During six attempts, about 2.6 Mm^3 sediments were released from the reservoir. Based on this experience, measures were taken to lower the amount of sediments in the reservoir. The amount of

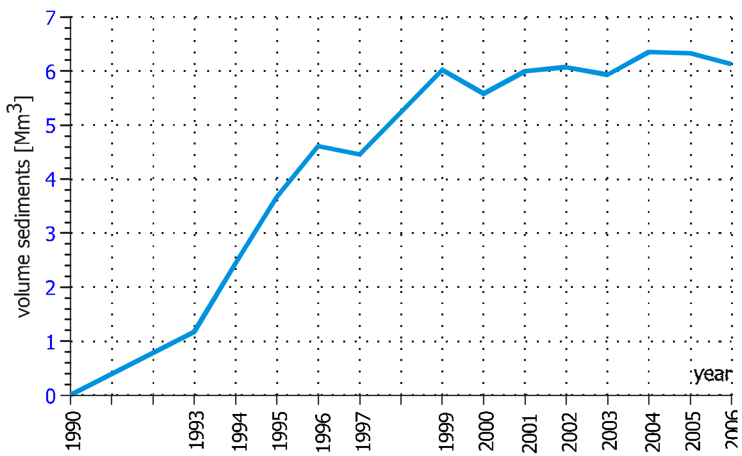


Fig. 9.4.5 Time course of sedimentation in the Bakaru reservoir (after Susilo, Nugroho, 2008)

sediments should be kept within acceptable limits by using the upper and lower outlets appropriately.

The problems with silting are encountered practically on every reservoir and it is possible to reduce this adverse phenomenon to a certain degree by an appropriate operating regime of the reservoir. By using the lower outlets it is possible to remove the sediments deposited near the dam body from the flood-control storage (Fig. 9.4.6). An example shows the release of fine sediments from the Angat reservoir. The operators of reservoirs, especially if the reservoir is used as a source of water, are not happy to see such an "uneconomical" release of water.

Fig. 9.4.6 Release of reservoir sediments (photos by P. Bláha - 2006 and 2010)

The average values of transported and settled coarse sediment load, particularly sand and gravel, can be measured directly only with difficulty because one flood will bring into the reservoir more of this sediment load than water under the normal stage over several years. Accumulated muddy sediments can reach a thickness of up to a few tens of metres and are detectable or measurable even below the water surface using modern methods. It is possible to observe the shape of a studied bottom in a 3D display using instruments of new generation. The probe sends a beam not only in the vertical direction, but also an acoustic signal is generated into more directions. Figure 9.4.7 shows a view of the bottom of the Dalešice reservoir using 3D sonar. A longitudinal groove can be seen in the middle of the image, which can be interpreted as the depleted area of

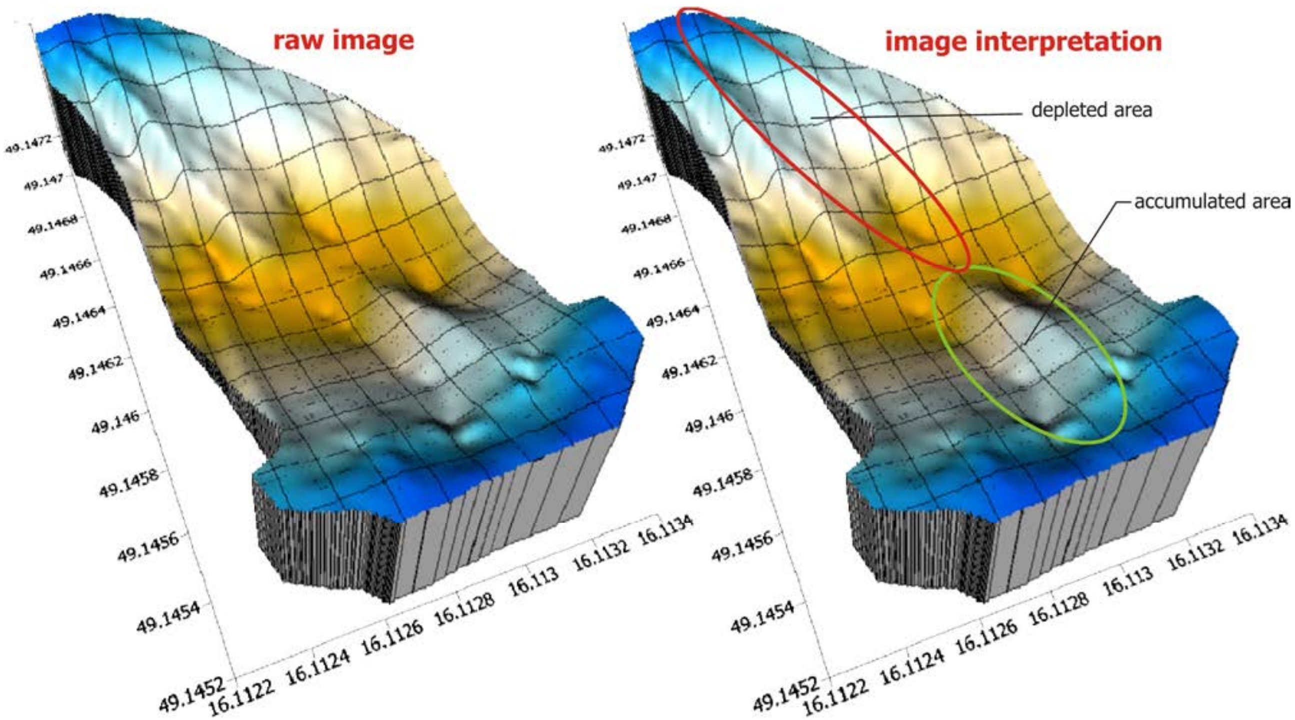


Fig. 9.4.7 Reservoir bottom view by 3D sonar (with a landslide)

a slope failure. This landslide formed in all likelihood after the area had been filled by a new dam lake, namely for two reasons. The entire area of the dam lake was subjected to engineering-geological mapping before it had been filled and no such a striking landslide was identified at that time. The second reason is the large distance between the depleted area and the accumulated area of the landslide. It can be explained only by practically zero friction of the slipping material and its lightening by water. It can be assumed that the rock material moved because its strength had been reduced by tectonic fracturing and weathering.

Great problems arise if the bottom of a reservoir before it was filled had not been cleared of trees, shrubs and other vegetation. Gradual decay of these materials reduces the level of oxygen in the water, which can be a cause of great losses when drawing out fish from the reservoir. The anaerobic products of decay include particularly hydrogen sulphide, which is very harmful for aquatic organisms; in addition, it is very dangerous for turbines, which it corrodes. Other products of decay are methane and carbon dioxide, which increase the negative effect of greenhouse gases.

A great danger for an artificial water reservoir can also be organic materials or fertilizers carried there from field fertilization in the river basin above the reservoir. On the one hand, decaying organic materials bring benefits to fisheries development, on the other, they are the cause of the growth of aquatic plants and the advancing process of eutrophication, which then greatly hampers the planned use of the reservoir, such as for recreation, use of water for drinking purposes or for irrigation economy. Therefore, before being filled, reservoirs for drinking purposes are also cleared of vegetation not only up to the limit of submersion, but also above it. Floated or loosened tree trunks can be a great danger for hydropower structures if not removed before the reservoir was filled. A cautionary example is an accident at the hydroelectric power station of the Sayano – Shushenskaya dam in Russia, where a large tree trunk probably reached a turbine and caused a true disaster (Fig. 9.4.8).

Fig. 9.4.8 Accident at the Sayano-Shushenskaya dam, 17 August 2009 (a photo by Reuters)

A serious problem is the regulation of discharges in deltaic sediments at large water reservoirs. Deltas and their degradation become developed in the end areas of large water reservoirs. Due to their development, the regime of surface water and groundwater, the character of landscape, the vegetative and soil cover change in the upper part of the reservoir. Deltaic sediments are accumulated materials of different origins, including organic matter. They become overgrown by vegetation and the soil is subjected to backward erosion. Deltaic processes at large water reservoirs take place worldwide; they were described in North and South America, Africa and also elsewhere and their occurrence has a global character. Newly formed hydromorphic deltaic landscapes create new land and littoral ecosystems with vast areas of aquatic vegetation and fauna. Attention of experts should focus on their study with the aim of their economic use and protection.

These processes at large water reservoirs were described on many dam reservoirs in the world, whether in dry arid areas (the Kapchagay water reservoir on the River Ili in Kazakhstan), or in humid areas of Europe (e.g. a cascade of water reservoirs on the River Dnieper in Ukraine) or on the Orava reservoir in Slovakia. One of the most representative examples of the formation of new deltas is the Kapchagay water reservoir in the south-eastern part of Kazakhstan. The reservoir with a volume of 28.1 km^3 designed for power generation and irrigation was commissioned in 1970. According to long-term measurement, up to 11 million tonnes of sediments are settled in the reservoir every year. These sediments in the end area of the reservoir gradually formed new islands with a strange hydromorphic character of landscape. The subsequent extension of islands and their gradual merge have led to the formation of a vast delta, which has been called the Kapchagay delta. The velocity of the process of the delta formation depended on the variability of the height of the water surface in the reservoir, on the extent and type of transported sediment load in relationship to the size of precipitation and hence on the carrying capacity of water. Figure 9.4.9 shows the time sequence of the delta as recorded in the images from the LANDSAT satellite. The black-and-white images are supplemented by a colour image from the GoogleEarth server dated 15 September 2009.

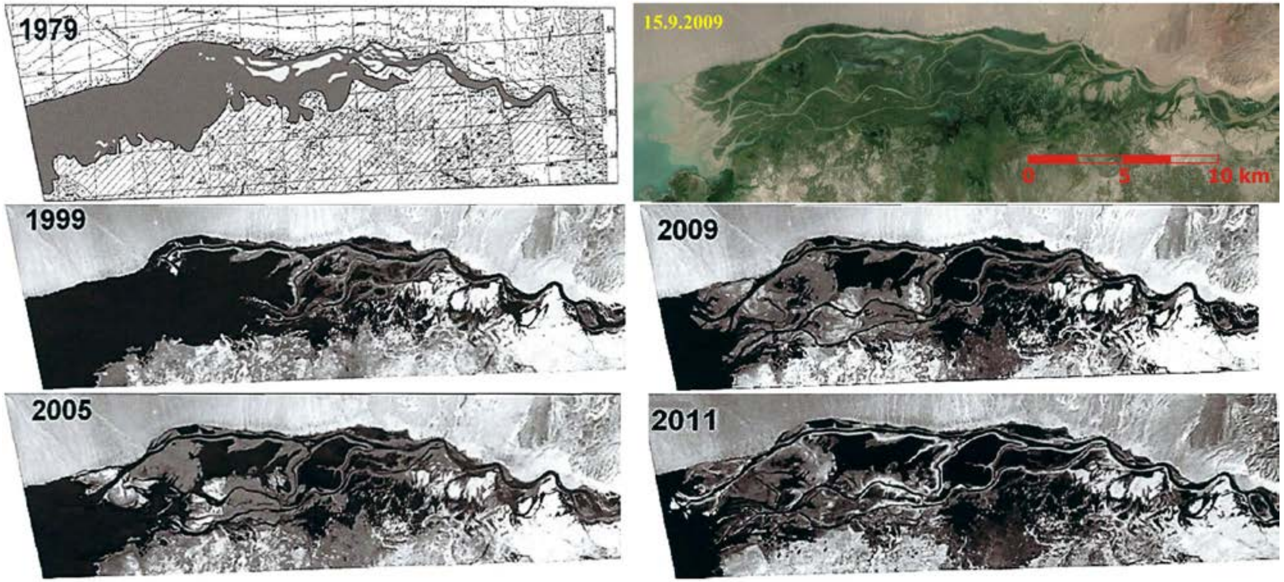


Fig. 9.4.9 Time course of "Kapchagay delta" formation (after Starodubtsev, Bogdanets 2008 + © GoogleEarth)

An interesting feature in such formed deltaic sediments and partially also on the river above the delta is the occurrence of backward erosion devastating and carrying away deltaic sediments downstream into the reservoir itself. If this is a reservoir for a pumped storage hydroelectric plant or a reservoir with a large up-and-down fluctuation of the water surface, this process of backward erosion becomes stronger.

Reservoir silting is relatively small in certain landscapes, whereas elsewhere it is so intense that it literally threatens the operation of the reservoir. Many cases were recorded in the history of dam reservoirs, where a water reservoir was fully silted up and put out of operation within the first tens of years. It is, therefore, necessary when carrying out a survey of a river basin to pay particular attention to the issues of reservoir silting, which chiefly means to carry out engineering-geological mapping of the entire periphery and to assess from which areas, both on the slopes above the reservoir and in the upper reach of the river and its tributaries, sediment load will be brought in, and what their character and assumed amount will be. Based on this survey and on the assessment of other aspects, it is then possible to prepare a plan of measures to reduce the amount of sediment load, for example by damming up gravel-bearing torrents flowing into the reservoir, by appropriate afforestation of slopes, by remediation of landslide areas, by bank lining as a measure against abrasion, or by implementation of other forest-engineering measures as well.

Recently, mathematical modelling begins to be applied even in this issue. This procedure can be used to model not only silting itself, but also all measures which should reduce silting. Suction dredgers, bypasses and other devices begin to be used for washing out sediments from the bottom of reservoirs. In an effort to reduce the adverse effects of transported material in a reservoir, great attention also begins to be given particularly to the design of turbine blades with the aim to reduce considerably the necessity of their frequent repairs or even their replacement.

9.5 Examples of Phenomena Affecting the Banks of Water Reservoirs

Today, the construction of a dam can no longer be considered as an exceptional project. At present there are at least 157 dams higher than 150 metres in the world (wikipedia.org). The true number is certainly greater because a number of dams are not listed in the wikipedia review. There are various criteria that can be used to assess the size of a dam. Dams can be ranked in terms of height or length, by the area of the reservoir, by the volume or length of the dammed reservoir or by the installed turbine capacity. The highest dam, Rogun, which is under construction on the River Vakhsh, will be 335 metres high. According to the information available, it is not yet finished, although the first turbine was put into operation at the beginning of 2008. The highest completed dam (300 metres) was built at Nurek on the same river and has been in operation since 1980. The world's longest dam at Hirakud on the River Mahanadi is 25.8 km long. The longest lake (660 km) was created by the construction of the Three Gorges dam on the River Yangtse. This dam also has the highest installed turbine capacity (22.5 GW). Lake Volta on the River Volta has the largest reservoir area (8,502 km^2) and the lake with the largest volume (180 km^3) has been created behind the Kariba dam on the River Zambezi. This dam caused 20 earthquakes stronger than 5 on the Richter scale due to the change in stress caused by the load of water when the reservoir was filled.

Many dam engineers and experts today claim that all the sites suitable for construction of a large dam have already been used. Nevertheless, in the world today, the impetus for the construction of new dams is increasing rather than gradually decreasing. This is most evident in the developing economies of Southeast Asia. According to the proceedings of the international conference "Asia 2008 – WRRED", 188 dams are under construction or being planned in China, and 78 in India.

Despite all human efforts and ingenuity, however, it was nature that created the largest dam in the world. After an earthquake in 1911, a dam 550 to 700 metres high was created by a rock fall on the River Murghob (Fig. 9.5.1). Lake Sarez, 65 kilometres long with a volume of 16.5 cubic kilometres, has progressively formed behind the dam that has been full for the last 25 years. One of the hazards associated with this dam is a landslide lying just behind the right flank of the dam. The landslide has a volume of 0.9 km^3, raising fears that if it suddenly collapses into the lake, the dam will spill over and be destroyed. Pessimistic estimates suggest that there may be as many as several million casualties if this catastrophe takes place.

After they were completed and commissioned in the 1950s and 1960s, the gigantic water reservoirs on the Rivers Yenisei, Angara, Ob, Amur, and the other rivers belonging to the Siberian cascade of hydroelectric power plants, were soon strongly affected by geodynamic processes. Artificial water reservoirs on the Yenisei and Angara are sometimes described as inland seas. This comparison is justified because they are similar in size to large inland lakes in Canada and Africa. The construction of water reservoirs on the Volga and Dnieper in the European part of the former USSR also had a harsh impact on the natural environment and the pattern of natural geological processes has been changed in significant ways.

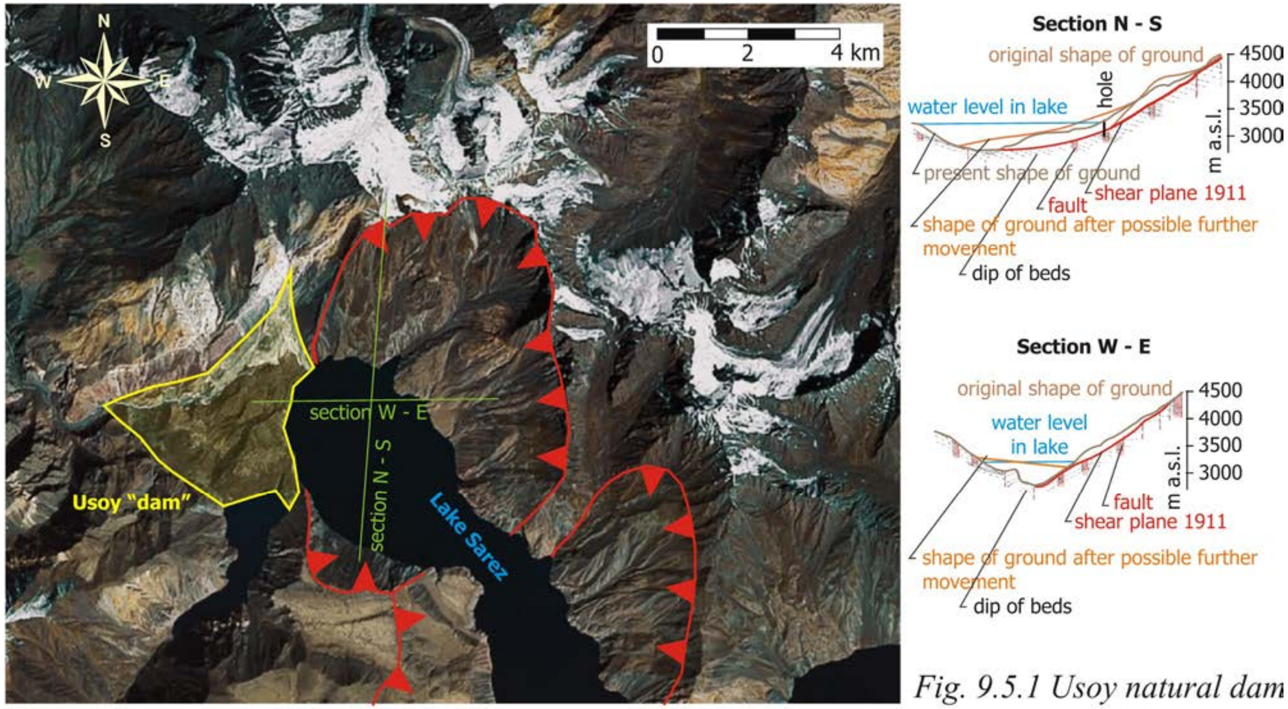


Fig. 9.5.1 Usoy natural dam and landslides on its banks (adapted from www.googleearth.com and Lim, Aklodov, 1998)

Newly created agricultural cooperatives and industrial enterprises are frequently sited near water reservoirs, i.e. in those areas near the banks that are exposed to the effects of erosion and the risks of slope failures when the groundwater level rises soon after hydro-engineering structures are put into operation. The geodynamic processes initiated on the slopes adjacent to new reservoirs are a cause of major problems that are hard to solve. Engineering geologists, hydrogeologists, geomorphologists, hydrologists and other specialists thus face a whole range of new challenges that demand the highest level of professional skill and responsibility. For these reasons, as early as the 1950s, research was already being undertaken in the following fields:

- Causes and effects of natural geodynamic processes induced by artificial water reservoirs that disturb the natural dynamic balance of the landscape;
- Studies of the processes by which a new balance of natural conditions is established after the disturbance caused by commissioned water reservoirs;
- Predictions of the development in space and time of anthropogenic geodynamic processes;
- Assessment of the scale of recession of the shoreline due to erosion, slope failures and other factors;
- Detection of the rate and size of changes in the banks and slopes around reservoirs, and the depth to which they reach below the water surface, the formation of cliffs, abrasion platforms (terraces), etc.;

- Determination of the effect of the fluctuation of the water level during operation on the development of geodynamic processes;
- Rational criteria for the selection of suitable sites for the construction of industrial and agricultural facilities and the adoption of suitable remedial measures for stabilization of reservoir banks; and
- Proposals for practical methods of water supply and ecological measures for preventing the pollution of dam lakes by contaminants and natural sediments.

The simple enumeration of the tasks listed above shows the wide range of issues which had not been dealt with previously and which have become very serious problems affecting the continued operation of water reservoirs. However, the progress that has been made in studying these phenomena marks a milestone in the solution of the problems affecting the reservoir areas of dams and has enormous theoretical and practical significance. This approach has enabled certain geodynamic processes brought about by human activity to be monitored and interpreted from the very beginning.

In the former Czechoslovakia, the changes affecting the banks of reservoirs were studied from the theoretical and experimental point of view. Major contributions to research were made by the Laboratory of Engineering Geology and Geotechnics of the Czechoslovak Academy of Sciences in Prague, by Stavebni Geologie in Prague, and by Geotest, which was the coordinator of the research project “Bank Changes Reservoir Areas of Dams” until 1982.

In a number of countries, the problems affecting the stability of the banks of water reservoirs were dealt with only as they arose or when a state of emergency had arisen, sometimes only after a catastrophe (Vajont, 1963). In Spain, the problems of the degradation of the banks of artificial water reservoirs were dealt with in particular by A. García Yagüe who studied landslides at the following dams: Bubal on the River Gallego, Arenós on the River Mijares, Correpoco on the River Saja and Yesa on the River Aragón (2000).

During survey work for the Yesa dam that was 76.5 metres high and completed in 1960, very slow creep movements were observed on the slopes above the future reservoir. On the right bank they have not been designated as dangerous; they have been monitored continuously up to the present day and do not pose any threat to the reservoir. In 1980, on the left bank, an extensive landslide was formed on rotational shear planes, and at the beginning of this century, another landslide of similar character took place (Fig. 9.5.2). Landslides pose a threat to the keying of the dam on the left bank and there is a risk that material will fall into the reservoir. According to some experts, this could trigger a wave and overspill of water across the dam. If such an event occurred, about 5,000 inhabitants of the village of Sangüesa below the dam would be affected. In Spain, this case is considered as alarming and has led to questions about the stability of banks in other reservoirs as well.

Huge problems, particularly ecological impacts, have arisen after the completion of the “Three Gorges” dam on the River Yangtze in the province of Hu-bei in central China. This is one of the largest dams ever constructed in the world. The gigantic dam was equipped with turbines to supply electric power to Shanghai, but its main purpose is to prevent the devastating floods that caused almost a half a million casualties in the last century alone. From the point of view of the development of power engineering, it is indisputably the largest construction in the world; the powerhouse with 26 generators has an installed capacity of 18.2 GW. In addition, an underground power plant has been designed where another six generators with a total capacity of 4.2 GW will be installed in the turbine room. However, the project has not met with universal approval because the rare ecosystem in the area has been affected and changes have taken place in the local climate.

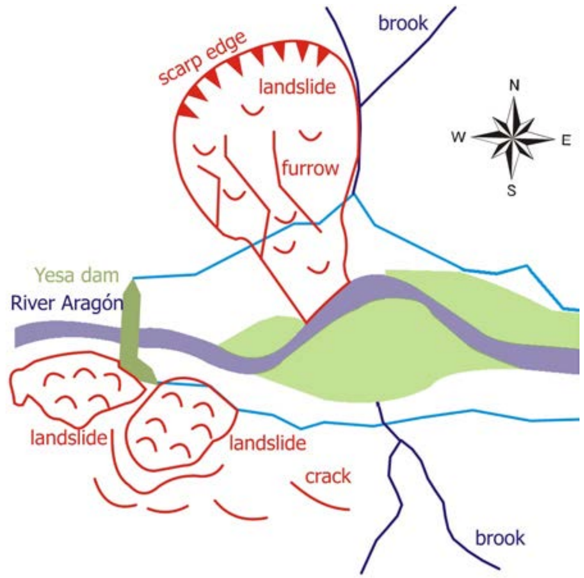


Fig. 9.5.2 Landslides on the Yesa dam

The water reservoir that has been created is 660 kilometres long, 1.1 kilometres wide on average and reaches a depth of 170 metres. 13 large cities, 140 towns and 1,352 villages have disappeared underwater. Most of the total population of 1.3 million inhabitants has been resettled in less fertile areas at higher elevations. This has led to the discharge of wastewater into the reservoir and to erosion and transfer of sediment to the reservoir caused by the cultivation of less fertile land higher on the slopes. During the rainy season, enormous deposits of mud and ecologically harmful waste will be washed from higher elevations down into the reservoir and the area around the port. This will not only cause an ecological crisis, but the sediments will also interfere with water transport. Some of the new towns for displaced persons were, unfortunately, located in areas of active landslides and it was necessary to relocate about 70,000 people and 1,500 businesses and agricultural cooperatives. The dam was put into operation in 2006 and in 2007 there were 91 landslides and, over a distance of 36 km, the works already carried out to stabilize and prevent erosion of the slopes were destroyed. The Chinese government has already expended CNY 120 billion (about USD 17.6 billion) on remediation, and plans to spend several hundreds of millions more on the prevention of ecological disasters caused by pollution and silting-up of the reservoir.

Analogous problems had already been encountered at the water reservoir of the Sanmenxia dam on the Yellow River in China. This dam, completed in 1960, should have helped to stop recurring catastrophic floods. Instead, it has slowed down the flow of the river and increased the deposition of fluvial sediments; as a consequence there are still floods on the Yellow River. In addition, the free transfer of fertile fluvial sediments to lower tracts on the river has been prevented and, as a result, agriculture has suffered.

The examples described above show that the construction of gigantic water reservoirs does trigger destructive geodynamic processes and irreversible ecological changes. These have a serious effect on the life of the human population in those areas. It is therefore necessary, now more than ever, for governments, planners and designers to be aware of the risks inherent in such large projects. It is the moral and professional duty of specialists in engineering geology to draw attention to the dangerous consequences before dams of such large scale are constructed.

Another reservoir threatened by a slope failure is the Akhangaran reservoir, which is affected by the Verkhneturkskiy landslide (Fig. 9.5.3). The first movement of this landslide was recorded in 1954. In 1955, the construction of the Akhangaran dam was begun. The keying of the dam on the left bank was close to the landslide itself. In 1969, the movement of the landslide was renewed and it had already reached the

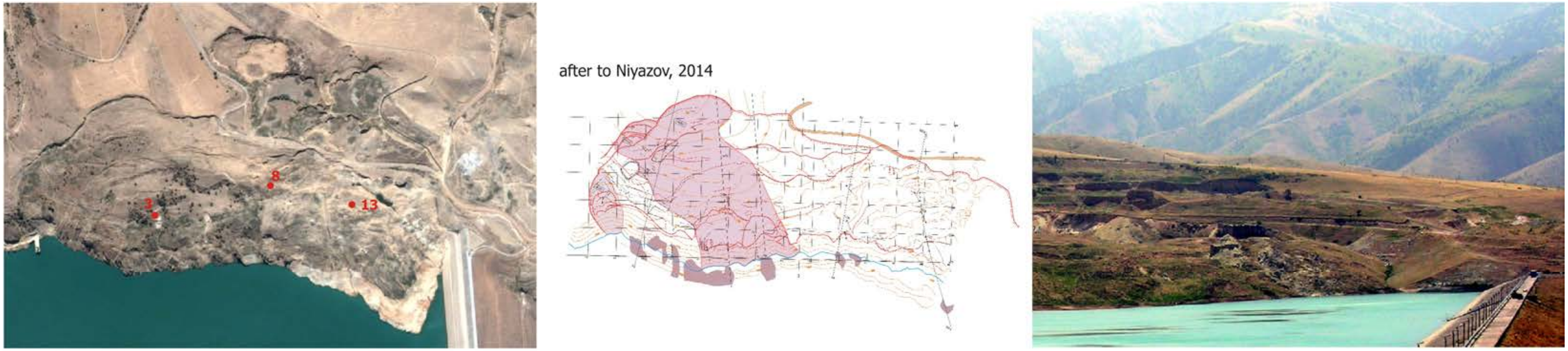


Fig. 9.5.3 Landslide "Verkhneturkskiy" on the bank of the Akhangaran dam, left – a Google Earth satellite view,right – a view of the left keying of the dam (a photo by P. Bláha - 2008)

construction site. The dam was completed in 1970 and the lake behind it has a volume of 200 Mm^3 when fully operational. The reservoir serves as water storage for the irrigation of cotton plantations and vegetable fields.

In 1979, the movement of the landslide was renewed again. The landslide has a total volume of 20 million cubic metres and is divided into an eastern, a central, and a western part. The central and western parts are most active. The movement of the landslide reached 32 to 36 metres. The greatest movements are recorded when the water in the reservoir reaches its maximum level. A remarkable fact is that 750 metres downstream from the foot of the dam, a giant 320-metre-deep opencast lignite mine begins, from which up to 3 Mm^3 of lignite are mined annually. So far as is known by the authors of this book, tests which would determine how much of the water percolating into the opencast mine is seepage from beneath the dam and how much is natural inflow of groundwater have not yet been carried out.

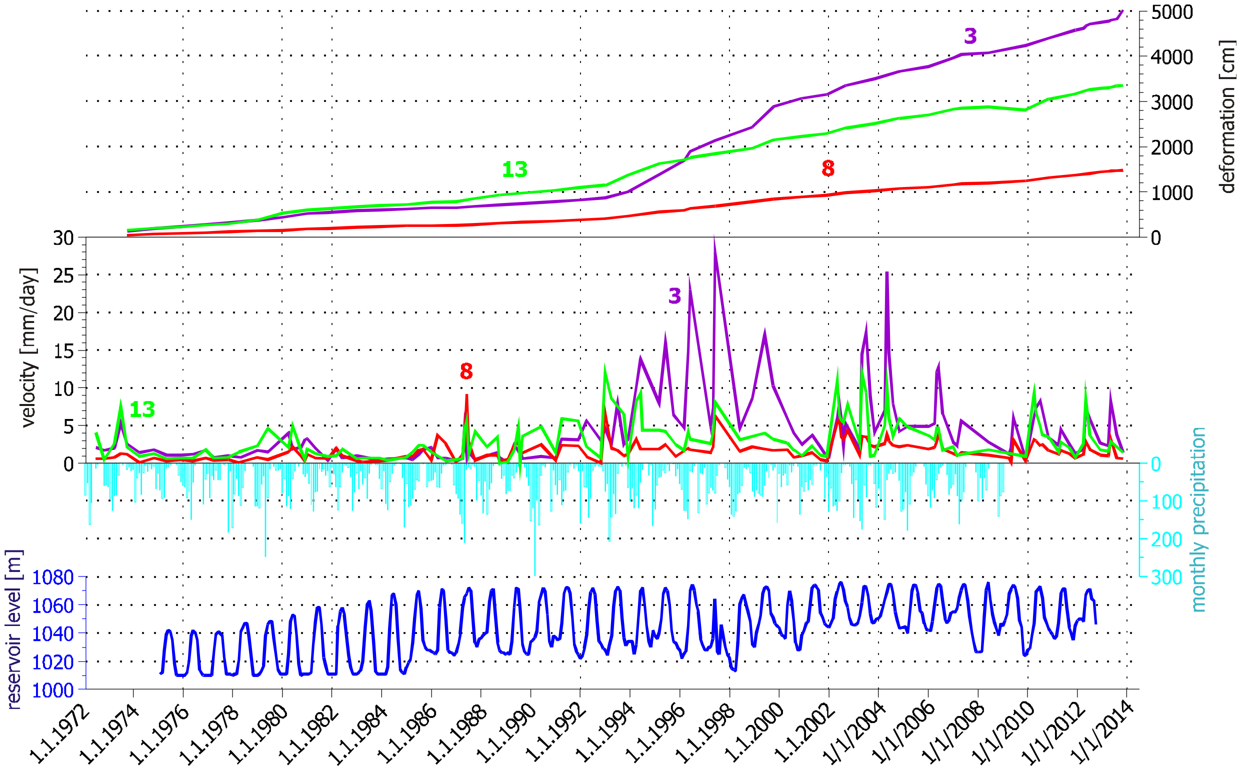


Fig. 9.5.4 Movement of the Verkhneturkskiy landslide (after Niyazov 2014)

At the present time, a plan to raise the dam by up to 20 metres is being considered and one of the designs allows for the excavation of the landslide, using 50 % of the volume extracted as construction material for raising the height of the dam.

The behaviour of the landslide has been monitored throughout its existence. The data available to the authors are not complete, but they cover the period from 1972 to 2002 (Fig. 9.5.4). The graphs of the velocity of the movement of the landslide show a visible difference in the behaviour of the landslide in the period up to 1985 and after this date. This difference also corresponds to a change in the regime of operation of the water reservoir. The information available does not explain why this change took place in 1985. It is difficult for an inhabitant of Central Europe to accept that plans are being made to raise the height of the dam when there is an active landslide immediately behind it. The question is whether the excavation of the landslide will stabilize the slope or whether a new slope failure will be formed higher on the slope. The lower part of the slope modified by the proposed excavation will be stable because the shear plane of the existing slope failure corresponds with the lithological boundary between the Quaternary cover and the solid basement.

Fig. 9.5.5 Silted tunnel at the Angat hydraulic facility (illustration provided by the company MWSS)

Fig. 9.5.6 Renewed growth of the vegetation cover after the typhoon Nanmadol (photos by P. Bláha - 2005 and 2006)

One of the problems that had to be solved in recent years is to ensure a supply of water sufficient to fill up a dam reservoir. Because of the ever-increasing consumption of water, it is common that a new reservoir is filled up not only from the basin of the river on which it is built, but that it is also recharged from another basin. Water is mostly transferred through a tunnel that is of dimensions large enough to cope with floods. A new problem has also arisen. This concerns the measures required for the safe and trouble-free operation of these diversion tunnels. The Angat dam provides an illustration of what can happen if basic preventive measures are neglected. Water is transferred along a 13-kilometre-long tunnel into the Angat reservoir from the basin of the parallel River Umiray. During the heavy rainfall connected with the typhoon Nanmadol in December 2004, the structure of the inlet to the hydraulic plant and the small hydroelectric power plant at the outlet of the tunnel were destroyed and the whole volume of the 13-kilometre tunnel was silted up (Fig. 9.5.5). The ballast consisted of sand, mud, and loose blocks of rock together with an abundant admixture of organic material. The absence of safety gates at the inlet of the tunnel was responsible for a four-month interruption in the use of water from the River Umiray; this was the time required to clean the tunnel and to put it into operation again.

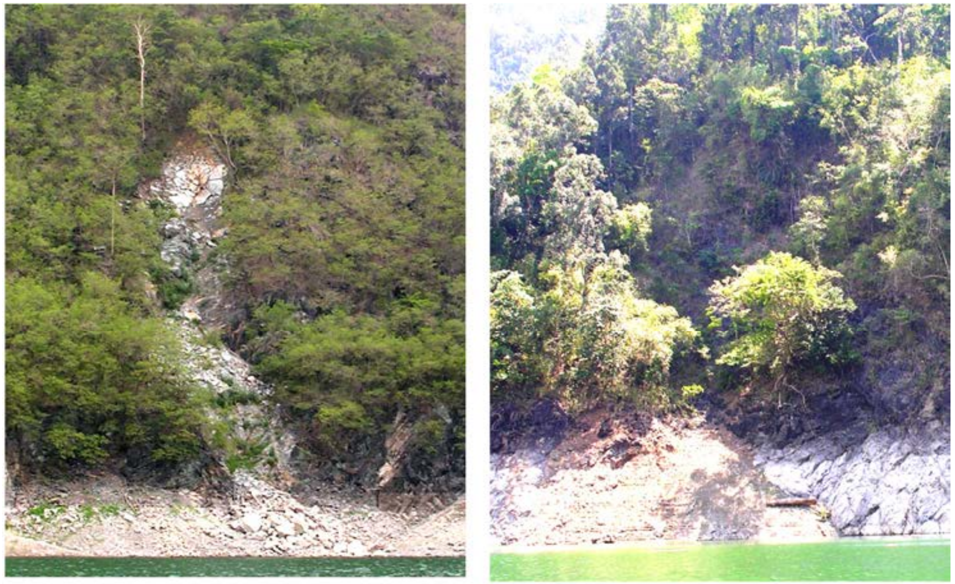


Fig. 9.5.7 Vegetation cover on slope failures (photos by P. Bláha - 2006 and 2007)

Nature itself is a significant factor in fighting degradation of the banks of reservoirs in tropical regions. The ability of the tropical vegetation to healing up various scars caused by water erosion or by slope failures in the reservoir area is astonishing. An example of how fast damaged banks can be covered by new vegetation is again from the Angat reservoir. Pounding by waves driven by the wind of the typhoon Nanmadol (up to 320 km/h) practically destroyed all the vegetation and a major part of the Quaternary cover on the banks of the reservoir as shown on the left side of Figure 9.5.6. After a year and a half, most of the damaged areas were covered by new vegetation as shown on the right side of Figure 9.5.6. It is usual for re-vegetation of denuded areas in the tropics to take place rapidly and numerous examples of this have been observed on different occasions in other regions.

The recovery of the vegetation cover occurs not only on the banks exposed by abrasion, but new landslides and/or exposed planes of failure are also covered with new vegetation in a very short time. The speed with which exposed areas are covered by new vegetation is more or less the same as in the case of damaged banks. The appearance of the scar caused by a rapid slope failure transitional between sliding and falling is shown on the left side of Figure 9.5.7; the revegetation of an older landslide at another site on the reservoir is shown on the right side (9.5.7).

Fig. 9.5.8 Landslides below the line of the maximum water level (a photo by P. Bláha - 1984)

The situation is quite different in arid climates. Very large differences between the maximum and minimum water levels are typical of reservoirs in arid regions, particularly if the water is used for irrigation. It is not exceptional for a reservoir to be empty at the end of the growing season. In such cases, the signs of slope failures are visible for years. The first example is shown in Figure 9.5.3, where the plant cover has not concealed any of the topographic features of the landslide over the 54 years since it took place. The same holds true when a landslide occurs below the line of the maximum water level. Even in this case, such a slope failure can remain visible for many years. Figure 9.5.8 shows an example from the Andijan reservoir. This photograph also illustrates the formation of terrace steps (terracettes), another phenomenon that affects the banks of reservoirs.

The formation of terracettes on unstable reservoir banks is quite common. It was observed not only in Uzbekistan, but also in the Philippines, in Bulgaria and in Czech reservoirs (Nechranice). Other illustrations are given in Figure 9.5.9. In the photograph on the upper right, this phenomenon is visible over a large vertical range at the Angat reservoir, where the small terrace steps have been cut into volcanic rocks affected by tropical weathering. The photograph on the upper left shows a section of the bank of the same reservoir; in this case blocks of unweathered rock, exposed by abrasion, rest on slopes into which parallel terracettes have been cut. In addition to the terracettes, the photograph on the lower left shows tree trunks lying on the banks of the reservoir. No organic matter or deposits should be present in reservoirs used for drinking water. Trunks of tropical trees should be removed from the surface of the reservoir and its vicinity. In this case, the natives living beside the reservoir use the tree trunks as a source of firewood. The photograph on the lower right is a view of the bank of the Charvak reservoir, where the annual fluctuation in the level of the water reaches up to 80 metres. The vertical height of the steps between individual terracettes depends on the period during which the level of the water in the reservoir remains at a given elevation and on the force and direction of the wind. The stepped terracettes formed on the banks of reservoirs are a relatively durable feature. At the Angat reservoir the terracettes were not even destroyed by the pounding of the waves caused by

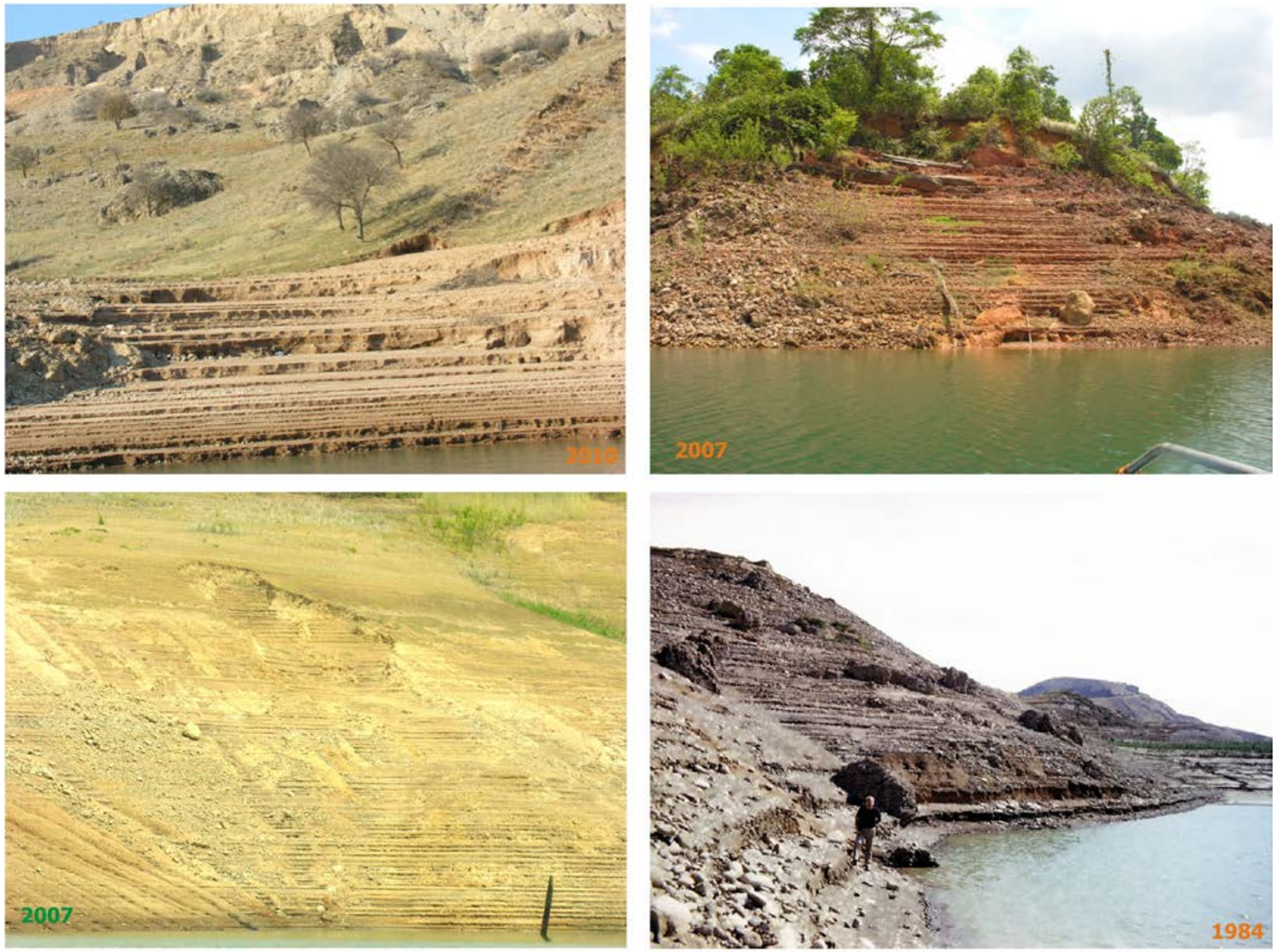


Fig. 9.5.9 Typical morphology of "terracettes" on reservoir banks (photos by P. Bláha)

the typhoon Nanmadol. The explanation for this lies in the fact that the reservoir was rapidly filled to the maximum level so that most of the stepped terraces formed on the bank lay underwater. In this case, the breaking waves could only cause abrasion and erosion on the bank in the vicinity of the high water level.

A common problem encountered in developing countries is human habitation on the margins of reservoirs. In Europe, strict regulations ensure that anthropogenic effects are practically excluded from the protected zones of reservoirs. In the Philippines, such regulations are not enforced. It is estimated that tens, and perhaps even hundreds, of rustic human dwellings have been constructed on or near the banks of the Angat reservoir. It is not uncommon for sites on landslides to be chosen for such dwellings (Fig. 9.5.10). Because of the movement of material down-slope, areas with gentler gradients are formed on the otherwise steep slopes. These areas are attractive sites for building houses and also for cultivation, while the fish in the reservoir provide the local people with an important resource for subsistence. It is unavoidable that fuels and lubricants leak from the engines of their boats, resulting in the pollution of the water in the reservoir. Improvised floats on local fishing-nets are made from plastic containers formerly used for various liquids. The flow of domestic waste from the smallholdings on the reservoir banks shown in Fig. 9.5.10 can also be imagined. In practice, it is not feasible for the state and local authorities to manage the situation at Angat in accord with the regulations common in the European countries.

Fig. 9.5.10 Settlement on landslide next to reservoir (a photo by P. Bláha - 2006)

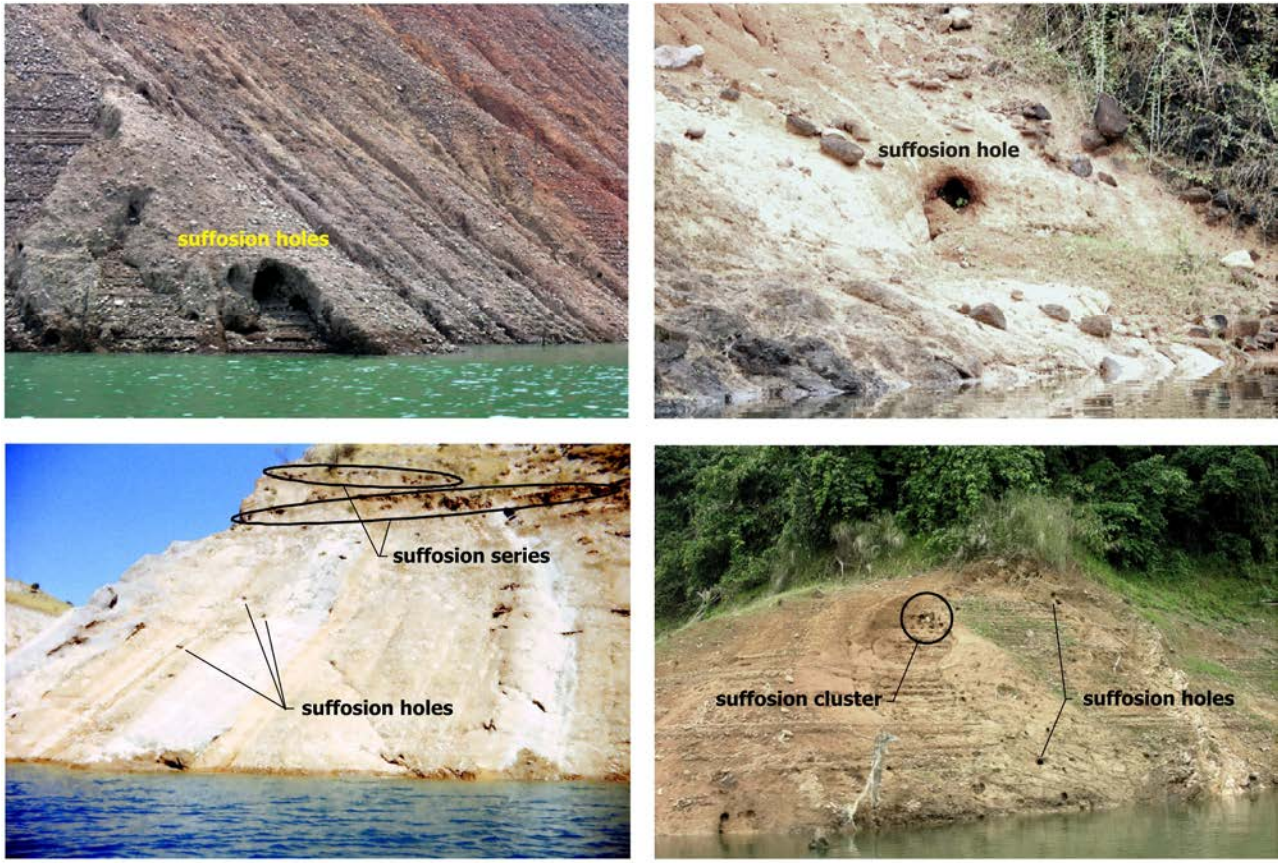


Fig. 9.5.11 Suffosion on the banks of the Charvak (left) and Angat (top and right) reservoirs (photos by P. Bláha - 1984, 2007, 2010 and P. Tábořík - 2010, top right)

Another phenomenon that is encountered on reservoir banks is suffosion. It is commonly observed in dams in many countries, both in humid and arid climates (Fig. 9.5.11). The photograph on the left shows the process at the Charvak reservoir. It can be observed how water flowing from suffosion holes creates sinter crusts. The photograph on the right shows the effects of suffosion on the banks of the Angat reservoir where both individual holes and clusters of holes can be seen. Suffosion phenomena have been observed to an increasing extent on the banks of reservoirs affected by slope failures. Because the stress on the rock mass is relieved as a result of slope failures, it is predictable that they become more susceptible to suffosion.

When the effects of erosion on the banks of reservoirs are being surveyed, situations sometimes arise when it is difficult to decide whether the cliff on a bank is the result of abrasion alone, or whether it is the scarp of a more extensive landslide exposed above the water level while the main mass of the landslide is concealed under the reservoir. In this case, the use of modern digital cameras and video recorders makes the interpretation of the morphological evolution of reservoir banks much more objective. If a continuous digital video recording is made during a survey of the banks of a reservoir, relevant sections of interest can be reviewed at any time subsequently and used in conjunction with evidence obtained from geological mapping and aerial photography to carry out a more effective interpretation of the processes responsible for shaping the banks. Successive video recordings of the same section of bank can be compared so that the changes taking place over time can be monitored and the processes responsible can be assessed objectively. Other valuable information can be obtained from an aerial survey of a reservoir. Photographs and video-recordings will provide some of the most important information gathered during a flight of inspection and the location of the flight path together with the time when the imagery was captured must be recorded simultaneously by means of a GPS.

Another method of measurement that is used in the survey of reservoir banks is sonar. Nowadays, sonar depth gauges coupled with GPS can easily be obtained at a reasonable price. If all these procedures are used in combination to survey the banks of a reservoir, it is possible to obtain all the information necessary to make a confident interpretation of the processes responsible for the modification of the banks. Sonar was used successfully by the Geotest team to interpret the processes responsible for erosion of the banks of the Angat reservoir (Fig. 9.5.12). Modern sonar devices enable the shape

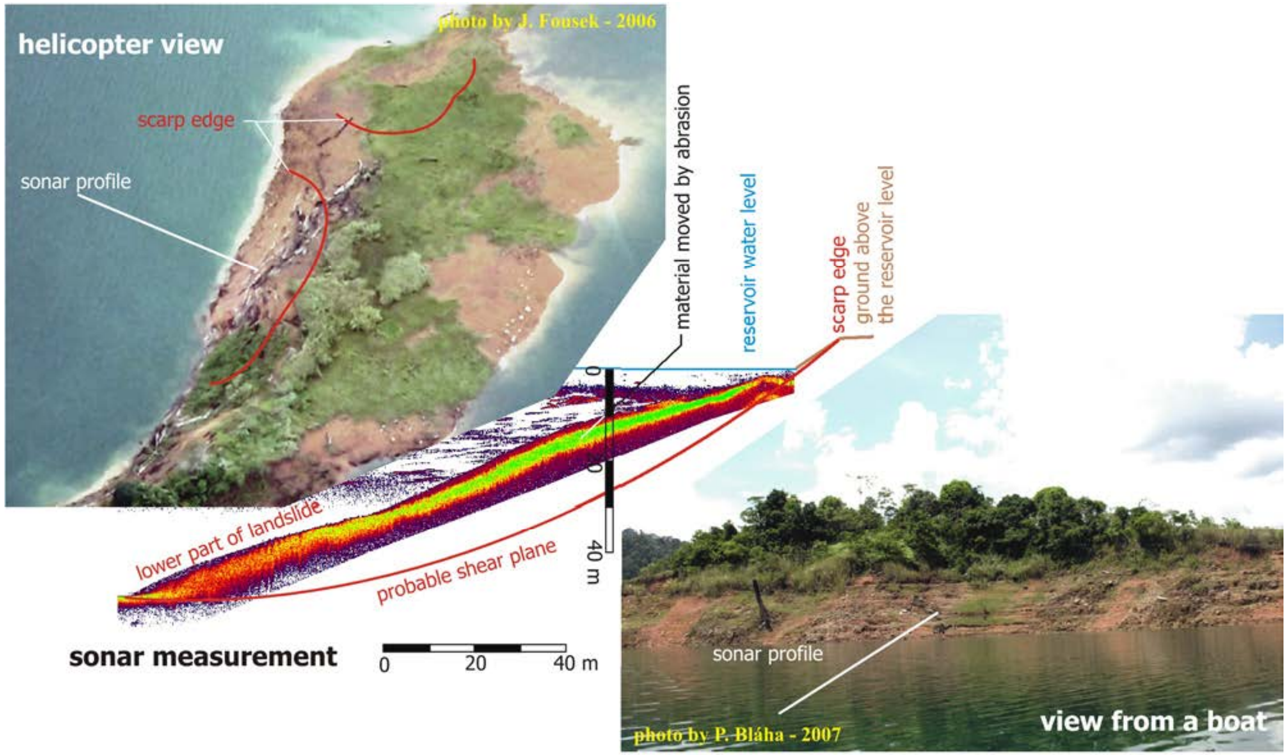


Fig. 9.5.12 Landslide on the Angat reservoir and sonar measurement

of the bottom of a reservoir to be determined, but they also provide information about its character. In the case of Angat, the sediments on the bottom of the reservoir are very mushy to a depth of about 30 metres. It can therefore be concluded that they have been formed largely by the washing out of Quaternary sediments on the shoreline that were subsequently transported to greater depths. The shape of the bottom at a depth of 30 to 50 metres shows that this part of the bank is affected by sliding. The lower part of the slope shows the definite bulge that is typical of the toe of a landslide. This, together with the shape of the main scarp determined by reconnaissance of the area from a helicopter, led to the certainty that this section of the bank had been shaped by a landslide.

Based on the observations and case histories described above, it must be concluded that slope failures in the reservoir areas of dams or in areas affected by the construction of a dam are often the cause of serious problems or, in certain cases, catastrophes resulting not only in great material damage, but also in injuries and loss of life (Vajont, 1963). An even more serious problem can be caused if a dam is created naturally by a large fall of rock or a landslide in a deep valley. On rare occasions, a dam formed by natural processes can create a natural lake and no catastrophe occurs during the period immediately after the valley has been blocked. The best-known case is that of the dam on the River Murghob (Fig. 9.5.1), where a strong earthquake of intensity M = 7.4 caused a huge rock fall in 1911. However, should the dam fail, there would be hundreds of thousands of casualties. This was the case in China in 1933, when an earthquake of intensity M = 7.5 triggered the collapse of more than 150 Mm^3 of material which blocked the valley of the River Min. More than 6,800 people died in the landslide and there were further casualties after the natural dam failed when another 2,500 people were drowned.

A similar accident happened on the Rio Mantaro in the Province of Huancavelica in Peru in 1974, when a rock fall of about one and a half cubic kilometres of material was triggered by heavy rainfall and erosion by the river. This created an earth dam 150 metres high. The landslide destroyed the whole village of Mayunmarca and 450 people died. When the area behind the natural dam began to fill up Peruvian experts realized that if the dam failed, there would be further casualties due to the flooding of the valley downstream. Overseas experts, including V. Mencl from former Czechoslovakia, were quickly called in and they recommended that the dam be bombed to avoid the more serious consequences of a natural failure. However, this was not successful, and later the dam failed, releasing a flood of water 28 metres in height with a velocity of 140 km/h. Villages were flooded and, inevitably, there were casualties. Because of this event, when the Czech team carried out a survey so that the hydroelectric power capacity on the Rio Mantaro could be expanded by building new facilities downstream, it was recommended that the important part of the structure be sited 30 metres above the surface of the river (1975). It was also recommended that the Peruvian authorities should pay serious attention to the problem of the instability of the right slope of the concrete dam on the Tablachaca reservoir, which forms part of the Mantaro hydroelectric power complex. The dam there is 75 metres high and sited on a landslide which poses a serious threat both to the stability of the dam itself, and to the water reservoir. Czechoslovak experts (Novosad, *et al.*, 1979) carried out an engineering-geological survey of the site and made proposals for remediation of the landslide.

10 Conclusion

It is certain that human society as we know it will not be sustainable in the twenty-first century without adequate supplies of water. One of the ways in which the supply of water for drinking, industrial purposes and irrigation can be guaranteed is by constructing dams. The evolution of knowledge and the development of technology mean that it is now theoretically possible to build dams under practically any natural conditions. However, the reality is that the more ambitious a project is, the more it will cost. It is therefore essential that the natural environment is utilized in a responsible and economic way, taking advantage of all the knowledge and experience that has been gained. This means that hydro-engineering projects will be undertaken only in favourable natural conditions, maintaining the highest standards of safety, and taking into account all the possible impacts on the environment while minimizing investment and operating costs.

The economic consequences of bad decisions, particularly at the stage of engineering-geological and geotechnical surveys, cannot be ignored. An unsuitable project and a bad plan for its construction arising from an inadequate knowledge of geological conditions can force a partial or complete change in the plan of work during construction. The costs of this will be considerable. In the extreme case, however, it can result in the deterioration of the construction site or dam during or after it is put into operation and irreversible damage to the environment. The rational and safe adaptation of the project to the natural environment is one of the most important tasks of the engineering geologist. Developments in knowledge will continue as a result of the synergy between geologists, climatologists, geomorphologists, ecologists and engineers, so that improvements in design and operation can take place in response to growing demands for the protection and improvement of the environment.

In the final decades of the last century, great progress was made in the evaluation of the rock mass as the environment for the construction of foundations. A range of new engineering-geological and geotechnical classifications have been developed, by means of which the rock mass can be divided into quasi-homogeneous units with similar geotechnical properties. They include, for example, engineering-geological classifications based on grain size, compactness, fracturing, degree of weathering of the rock, etc. Modern working procedures in civil engineering enable the parameters of the rock mass to be refined to the level required for survey and design purposes.

The development of computer technology has led to new working procedures for surveys and design. This applies not only to the methods used for making measurements and storing the data, but also to the techniques now available for processing and integrating large sets of data from different sources. Digital information is now acquired using a wide range of new instrumentation in addition to conventional geological procedures. The scale of observation ranges from remote sensing down to electron microscopy. It has been demonstrated that indirect geophysical methods can enable the more reliable interpretation of the geological composition and structure of a site, but they are also indispensable for planning the layout of direct exploratory workings and for determining the spatial variations in the properties of the rock environment by means of various correlations. New procedures for geotechnical tests can now provide designers with the values of parameters required by modern methods of design. Another major step forward in data collection, management, processing, evaluation and presentation is the introduction of Geographic Information Systems (GIS) and three-dimensional computer modelling in virtual space. The application of these techniques for the compilation and manipulation of data have provided new possibilities for the visualization and assessment of information obtained from archives and by observation and measurement at different stages of a survey. Today, it would be unthinkable to undertake a survey without the benefit of these state-of-the-art techniques.

When this book was being written, a decision had to be made about the best approach to take to the subject. The procedures used in engineering-geological and geotechnical surveys could have been described systematically giving practical illustrations of the use of each technique or the theory behind the application of particular methods could have been described in detail, omitting practical illustrations and case studies. In fact, the authors chose to use case studies as the principal theme for the book, relying on the fact that the readers would have at least some grounding in the theoretical background to geophysics and rock and soil mechanics. At the same time it was important to explain the importance of the systematic planning of each stage of a survey and the categories of information that should be obtained. It has been said many times over that observations made at first-hand in the field are never forgotten. It is appropriate to recall the old geological precept, frequently told to us by our teachers, “It is better to see once than hear twenty times”. For this reason, the illustrations in this book are in colour so that the reader can share, as nearly as possible, the experience of the original observer in the field.

The authors have given emphasis in this book to the fundamental principles governing the execution of an engineering-geological survey at a dam site. Attention has been drawn to the importance of the division of a survey into stages corresponding to the successive design stages, giving a detailed list of the individual tasks for each survey stage. An important prerequisite is the definition of the technical specifications and the planning of survey work in which the investor, the designer and the client who contracted the survey agree about the basic questions which the survey should answer.

The tasks to be undertaken when making an engineering-geological map of the area of a hydro-engineering project have been defined, and those relating to the dam site itself have been described in greater detail. The scope of both the preliminary and detailed hydrogeological surveys has also been described. The types of direct exploratory workings made by drilling, surface excavation and mining underground have been discussed. Attention has been focused particularly on the investigation of the area affected by the construction of the dam itself, and also on the particular conditions favouring the construction of different types of dams, especially gravity and concrete arch dams. Recommendations have been made for the average depths and spacing of exploratory workings in relationship to the category of the engineering-geological complexity of the site and the height and type of dam involved. These are useful guidelines for planning survey work but cannot be considered definitive. It has been emphasized that all proposals should be based on a thorough assessment of the geological and topographic setting of the site concerned. The criteria used to determine the size of the area for preliminary and detailed surveys have been discussed. In particular, the methodology used for the comprehensive documentation of exploratory workings has been described, including some new procedures used by both authors during their long practical experience in engineering geology and geophysics.

The general rules governing geotechnical surveys have been given and the basic procedures used in this work have been illustrated using practical examples. The fundamental types of field and laboratory tests used to determine the values of geotechnical parameters required for design purposes have been described and the correlations between the mechanical and physical properties of rocks, both causal and statistical, have been discussed.

Great attention has been given to the engineering-geological survey of the reservoir areas of dams. It has been emphasized that modification of reservoir banks by abrasion and slope failures can have a profound effect on dams and reservoirs and the surrounding environment. Clearly, monitoring and forecasting the development of these geodynamic processes is a necessary condition for the successful operation of most dams.

The book has been illustrated abundantly with examples taken from the practical experience of both authors. The methods used to solve the problems identified during the engineering-geological survey of dam sites were those in current use at the time when the project was being carried out. However, regardless of the time and circumstances, it must be emphasized that all surveys were carried out at the highest possible professional and technical level compatible with the existing state of knowledge and experience in engineering geology, geophysics and design, and the capacity of the equipment and technology available. This goal was certainly achieved in the surveys for Dalešice, Centro Cuba and elsewhere, during which practically all survey methods available at that time were used. The comprehensive processing of the results depended on successful collaboration between all the members of the team involved. The basic prerequisite for successful collaboration was the right choice of specialists from individual branches. But that alone did not guarantee success. It has been shown that individual specialists should also have a good knowledge of the underlying principles and current advances being made in fields related to their own areas of expertise. The synergy developed by the intelligent cooperation between members of the team responsible for a survey has led to surprisingly successful conclusions. By making full use of the combined theoretical knowledge and practical experience of a good team, an almost complete description of the geological composition and structure of an area of interest, together with credible values for the physical properties of the rocks, can be delivered to a designer.

Dams are certainly among the largest and most demanding structures in terms of their technical complexity. Every dam is in its own way unique because of the specific combination of natural and technical conditions that characterize each site. When planning a dam project, the technical and economic features of the design must be adapted as far as possible to geological and topographic conditions, bearing in mind the sensitivity of the environment. One of the main purposes of this book has been to describe the sequence of survey work that is necessary for hydrotechnical structures in order to rationalize and improve the quality of engineering-geological and geotechnical surveys. Even though each hydro-engineering project is set in a different natural environment and has different technical characteristics, the survey work should always be based on definite generally applicable procedures. There are, of course, an unlimited number of questions that can be asked about the composition and structure of any particular area. It would be absurd to assume that a perfect knowledge of the geology of the construction site and its surroundings could be obtained. The important thing is to find out as much as is necessary about the structure and the mechanical and physical characteristics of the rock environment, and its stability over time to carry out the construction of a dam safely according to best contemporary practice and the legal standards governing such work. Feedback from experience gained on other projects, both local and international, will also play a part. It would be wasteful to carry out investigations which are neither applicable to the construction or operation of an intended dam nor of any other practical value.

Global warming of the Earth has led to changes in climatic patterns. In particular, there is a noticeable increase in the frequency of rainstorms in various areas so that serious floods have taken place in recent years. The detrimental effects of flooding have been magnified by deforestation and intensive cultivation and grazing in the headwaters of many river basins. The capacity of soils, natural depressions and vegetation cover to moderate the run-off is progressively decreasing and, as a result, catastrophic floods are taking place more often. Dams have been built in a number of different countries, and other technical measures have been introduced to manage flood peaks. However, there are economic and natural constraints on the number of dams and reservoirs that can be built for the retention of water in future because most of the suitable dam sites in a number of countries have now been used. The topographic and economic limitations in many countries mean that major new dam projects will not be undertaken in the foreseeable future. The decades during which widespread construction of new dams took place, especially in industrially advanced countries, are now past. Attention will now be focused increasingly on the renovation and improvement of facilities that already exist and on integrated modern strategies for flood protection. One of the favoured measures is to build polders to control the release of flood water. These are normally dry reservoirs that capture or slow down a flood wave, thus reducing the threat of extensive damage to property and human life. The engineering-geological survey for the construction of polders will call for new approaches, but the experience gained in the construction of conventional dams will prove invaluable. It is the conviction of the authors that much of the experience gained in surveys for dams that have been described in this book will also be applicable to surveys for the construction of flood protection. Engineering geologists now face new challenges, but it is certain that the lessons learned through practical experience in the past will prove equally valuable in the future.

Abbreviations

A20M10N20B	Electrode array; 20, 10 distances between electrodes in metres	ÖNORM	Austrian Recommended Standards
CBP	Mise-a-la-masse method	P	longitudinal wave
CBR	Proctor standard test	pc	Pieces
CL	Caliper log	PER	Relative permittivity
CON	Specific conductivity	PL	Photometry log
CPT	Cone penetration test	PM	Pumping method
CR	Czech Republic	PSHEP	Pumped storage hydroelectric plant
CR	Cone resistivity	QST	Cone resistance
CRM	Modified classification of core recovery	QT	Total force acting on the cone
c_u	Cohesion	R16"	Resistivity log (short normal)
ČSN	Standard of Czech Republic	Rap	Resistivity log (short normal)
DEMP	Dipole electromagnetic profile	RM	Resistivity measurement
DP	Documented point	RQD	Rock quality designation
DTA	Differential thermal analysis	Rw	Water resistivity
E	Static modulus	S	transversal wave
E_1	Pressiometric modulus	SEM	Scanning electron microscope
EC	European Union	SL	Sonic (velocity) log
EC	European Commission	SP	Spontaneous (self) polarization
E_{def}	Modulus of deformation	SPT	Standard penetration test
E_{dyn}	Seismic modulus	SRP	Symmetrical resistivity profiling
EEC	European Economic Community	SSR	Shallow seismic refraction
EG	Engineering geological	T_a	Pattern of the Earth's magnetic field
EIA	Environmental Impact Assessment	TEM	Transmission electrone microscope
ERT	electric resistivity tomography	TIWAG	Tiroller Wasserkraftwerke, A, G,, (radial press)
FRL	Fluid-resistivity log	TM	Temperature log
FS	Skin friction	USBR	United States Bureau of Reclamation
FS	Unite sleeve friction resistance (or: local sleeve friction resistance)	V	Velocity
GGL	Gamma gamma log	V_b	Boundary velocity
GIS	Geographic information system	VES	Vertical electrical sounding
GPR	Ground-penetrating radar	VLF	Very low frequency
GPS	Global Positioning System	w	moisture
GR	Gamma ray log	W10	Wenner electrode array; 10 distances between electrodes [m]
GWL	Groundwater level	WPT	Water pressure test
H	Height of backwater	WS	Water structure
X	Dynamic resistance	NNL	Neutron neutron log

XRD	X-ray diffraction	NOC	Specific conductivity
HEP	Hydroelectric plant	α	Ratio of dam height to the width of the dam crest
ICOLD	International Commission on Large Dams	ϕ_{ef}	Angle of internal friction
ICS	International Commission on Stratigraphy	γ_0	Bulk density
I_γ	Dose rate (in air)	κ	Magnetic susceptibility
ISO	International Organization for Standardization	ρ	Bulk density
IUGS	International Union of Geological Sciences	ρ_a	Apparent resistivity
MSL	Magnetic susceptibility log	σ_{red}	Reduced strength

References

Classical:

1. ABDULLAEV S, – BLÁHA P, *Inženýrsko-geologický monitoring výstavby přehrady Rezaksay (Engineerig Geology Monitoring of the Razaksay Dam Construction),* Praha, EGRSE, 2012, Vol, XIX, No, 2, p, 1-6,
2. ABDULLAEV U, – ABDULLAEV S, – BLAHA P, – AKHMEDOV A, – KHUDOBAKHSHOVA,: The Usoy Dam, Lake Sarez and Possibilities of Geophysical Methods, *HYDRO 2011*, Praha, 2011,
3. ADAMUS, B, Fotodokumentace průzkumných štol (Photo documentation of exploratory galleries), In NEŠVARA, J, et al, *Hrhov – zpráva o inženýrskogeologickém průzkumu pro 2, etapu řešení*, Brno: Geotest, 1974, MS,
4. ALTOVSKIY, M, E, *Spravočnik gidrigeologa*, Moskva, 1962,
5. ANISHCHENKO, A, P,, DEGOVEC, A, C,, KARAMANOV, U, K,, KHEGAY, A, Y, Zashcita goroda Alma-Aty ot seley, (Protection of the city of Almy-Aty befor mudflows) In *Opolzni i seli (Landslide and mudflows),* Centre of International projects, Moskva, 1984,
6. ASTRAKHANTSEV, V, I, *Issledovanija beregov vodochranilišč*, Irkutsk: Akademija nauk SSSR, Sibirskoje otdělenije, 1972,
7. BARTON, N, – LIEN, R, – LUNDE, J,: Engineering Classification of Rock Masses for the Design of Tunnel Support, *Rock Mechanics*, 4, 1974,
8. BARTON, N, – LIEN, R, – LUNDE, J,, Engineering classification of rock masses for the design of rock support, *Rock Mechanics* 6, 1974, p, 189-236,
9. BELL, F, G, *Engineering Geology and Constructions*, Spon Press II, New Fetter Lone, London ECAP 4EE, 2004,
10. BIENIAWSKI, Z, T,, Engineering classification of jointed rock masses, *Trans, S, African Instn, Civ, Engrs,*, 1973, Vol, 15, No, 12, p, 335 - 344,
11. BIENIAWSKI Z, T,, 1976, Rock mass classifications in rock engineering, In *Proceedings Symposium on Exploration for Rock Engineering*; ed, Z,T, Bieniawski, Balkema, Rotterdam, 1976, p, 97-106,
12. BLAGA, P, Komplexnaja seismičeskaja dokumentacija skvažin, In *Izmerenije i racionalnoje ispolzovanije geologičeskoj sredy*, Tashkent: SAIGIMS, 1989, p, 55 – 68,
13. BLÁHA, P, Předběžný geofyzikální průzkum pro přehradní profil Slušovice (Preliminary geophysical survey for the Slušovice dam profile), In FOUSEK, J, *Inženýrskogeologický průzkum pro vodní dílo Slušovice*, Brno: Geotest, 1971, MS,
14. BLÁHA, P, Zpráva o měření na inženýrskogeologické mapě Dobrá-Janovice (Report on measurement on the Dobrá-Janovice engineering-geological map), In ADÁMEK, O, *Inženýrskogeologická mapa 1:25000 Dobrá-Janovice*, Brno: Geotest, 1975, MS,
15. BLÁHA, P, Výsledky geofyzikálních měření na lokalitě Tršice (Results of geophysical measurements at the Tršice site), In GURECKÝ, J, *Závěrečná zpráva o doplňujícím inženýrskogeologickém průzkumu pro vodní nádrž na řece Olešnici*, Brno: Geotest, 1978, MS,
16. BLÁHA, P, Karotážní měření (Logging measurement), Brno: Geotest, 1980, MS,

17. BLÁHA, P, Zpráva o geofyzikálním měření v prostoru sdruženého objektu PVE Dlouhé Stráně (Report on geophysical measurement in the area of the combined facility of the Dlouhé Stráně PSHEP), In PAPOUŠEK, Z, et al, *Zpráva o doplňujícím inženýrskogeologickém průzkumu za účelem zpřesnění poznatků o horninovém prostředí pro akci PVE Dlouhé Stráně – 2, ucelená část, sdružený objekt*, Brno, Geotest, 1981, MS,
18. BLÁHA, P, *Závěrečná zpráva o geoakustických měřeních na Jezerce (Final report on geoacoustic measurements on Jizerka)*, Brno: Geotest, 1986, MS,
19. BLÁHA, P, In Nijazov R, A, et al, *Monitoring exogennych geologičeskich processov*, Tashkent: Uzbekgidrogeologija – Gidroingeo, 1990, MS,
20. BLÁHA, P, *Geoakustická metoda při výzkumu svahových deformací (Geoacoustic method in a survey of slope deformations)*, kandidátská dizertační práce (candidate dissertation thesis), Ostrava: VŠB, 1990,
21. BLÁHA, P, *Geofyzikální metody při průzkumu a výzkumu svahových deformací (Geophysical methods in the survey and research of slope deformations)*, habilitační práce (habilitation thesis), Ostrava: VŠB, 1991,
22. BLÁHA, P, Seismická měření na přehradním tělese (Seismic measurements on a dam body), In *Seismologie a životní prostředí*, Ostrava: ÚGN, 1993, p, 85 – 92,
23. BLÁHA, P, Shallow refraction processing in geology, In *7th International IAEG Congress*, Rotterdam: Balkema, 1994, p, 61 – 68,
24. BLÁHA, P, *Seismická tomografie (Seismic tomography)*, Praha/Brno: NIS Geofond/Geotest, 1996, p, 1 – 50,
25. BLÁHA, P, *Inženýrská geofyzika svahových deformací v hornické a stavební geotechnice (Engineering geophysics of slope deformations in mining and building geotechnics)*, doktorská disertační práce (doctoral dissertation thesis), Ostrava: VŠB, 1997,
26. BLÁHA, P, Svahové deformace, kras a geofyzikální metody v okolí Hrhova (Slope deformations, karst and geophysical methods around Hrhov), In *Svahové deformace a pseudokras*, ČGS, 2004, p, 1–10, 17 images, [CD carrier],
27. BLÁHA, P, – NEŠVARA, J, Geofyzikální průzkum svahových deformací na okraji neovulkanitů pro VD Plachtince (Geophysical survey of slope deformations at the edge of novolcanic rocks for the Plachtince WS), In *Aplikace geofyziky v IGHG*, I, díl, Brno, 1976, p, 507–518,
28. BLÁHA, P, – NEŠVARA, J, Použití geoakustické metody při sledování vývoje horninové klenby ve vápencích (Use of a geoacoustic method in monitoring the development of the rock arch in limestones), In *Geotechnický prieskum pre tunely a navrhovanie tunelových ostení*, ČSVTS Vysoké Tatry, Dům techniky Košice, 1978, s, 205–210,
29. BLÁHA, P, – NEŠVARA, J, *Určení příčné seismické vlny a Poissonova čísla tortonských jílů na dálnici D47 u obce Komořany (Determination of a transverse seismic wave and the Poisson's ratio of Tortonian clayes on motorway D47 by the village of Komořany)*, Brno: Geotest, 1981, MS,
30. BLÁHA, P, et al, *Speciální metody geofyzikálních měření na sesuvech (Special methods of geophysical measurements on landslides)*, Dílčí zpráva o PÚTR 70,1852, Brno: Geotest, 1985, MS,
31. BLÁHA, P, – VLASTNÍK, M, *Informe final de las mediciónes geofísicas realizadas para el pro-yecto Alternativas de regulación de los ríos Guadiaro y Genal*, Brno, Madrid: Geotest, 1992, MS,
32. BLÁHA, P, – VLASTNÍK, M, *Využití metody pronikání při zpracování mělké refrakční seizmiky pro účely inženýrské geologie (Application of the method of penetration in processing shallow seismic refraction for the purposes of engineering geology)*, Geologický průzkum, 1992, 6, p, 169 – 171,
33. BLÁHA, P, – LINCER, L, Závěrečná zpráva o geofyzikálních měřeních na lokalitě Žamberk – Elitex (Final report on geophysical measurements at the Žamberk site – Elitex), In Henešová A, *Žamberk – ELITEX – doplňkový hydrogeologický průzkum*, Brno: Geotest, 1993, MS,
34. BLÁHA, P, – HORSKÝ, O, Engineering geophysics for dam site selection, In *7th International Congress IAEG*, Rotterdam: Balkema, 1994, p, 69 – 78,
35. BLÁHA, P, – KOTTAS, J, – SOCHOR, J, Geological inhomogeneities in seismic tomography, In *7th International Congress IAEG*, Rotterdam: Balkema, 1994, s, 79 – 87,
36. BLÁHA, P, – LINCER, L, – WOZNICA, L, Geoakustické monitorovací měření na přehradě Vír (Geoacoustic monitoring measurement at the Vír reservoir), In *Polní geotechnické metody 94*, Ústí nad Labem: Dům techniky Ústí, 1994, p, 23 – 26,

37. BLÁHA, P, et al, Závěrečná zpráva o geofyzikálním měření v trase dálnice Vyškov – Chropyně v kilometráži 33,5 – 35,4 (Final report on geophysical measurement in the route of motorway Vyškov – Chropyně at 33,5 – 35,4 kilometres, Brno: Geotest, 1997, MS,
38. BLÁHA, P, – MRLINA, J, – NEŠVARA, J, *Gravimetric methods in the exploration of slope deformation*, Praha: EGRSE, 1998, 1, p, 21 – 24,
39. BLÁHA, P, – MÜLLER, K, *Uplatnění geofyzikálních metod v geotechnice a stavebnictví (Application of geophysical methods in geotechnics and civil engineering)*, EGRSE, 2001, 1 – 2, p, 4 – 5, ISSN 1211 – 359X,
40. BLÁHA, P, – MÜLLER, K, The signification of geophysical monitoring in geotechnical practice, In *Publications of the institute of geophysics Polish academy of sciences*, Warszawa, 2002, p, 356 – 365,
41. BLÁHA, P, – MÜLLER, K, Application of Geophysical Methods in Geotechnic and Construction, Praha, EGRSE, 2003, 1 – 2,
42. BLÁHA, P, et al, *VEP – rozložení pole ve svahových deformacích (VEP – distribution of the field in slope deformations)*, Brno: Geotest, 2006, MS,
43. BLÁHA, P, et al, *Philippines – Drinking Water for Manila, MHPP, Report on a Geological Survey for Protective Measures in a Wider Vicinity of the Power Plant*, Brno: Geotest, 2007, MS,
44. BLAHA P, et al, Geoelectrical Survey for the Feasibility Study of Bawanur Dam Site,, *EGRSE*, XVII, 2, ČAAG, 2010, ISSN 1803 – 1447, s, 14 – 47,
45. BLAHA P, et al, Geoelectrical Survey for the Feasibility Study of Bawanur Dam,, *HYDRO 2011*, Praha, 2011,
46. BLAHA,P,, DURAS,R,, FOUSEK,, J,, HORSKY, O,(2011): An Orientation survey of the Bawanur Dam, *HYDRO, 2011*, Praga, Session 7: Hydro Development in Asia,
47. BLÁHA P, et al, *Měření abraze břehů vodních nádrží (Measurements of abrasion banks of reservoirs),* Výzkumná zpráva,, CREA, Brno, 2012, MS,
48. BLAHA P, – SKACEL B, Problems of the Angat hydro-engineering structure, *Water Resources and Renewablw Energy Development in Asia*, 14,06, The International Journal on Hydropower & Dams, Wallington, Aqua~Media International, 2012, on CD,
49. BLAHA P, et al, An engineering-geological survey for the Bawanur dam in Kurdistan, Water Resources and Renewable Energy Development in Asia, 09,11, *The International Journal on Hydropower & Dams*, Wallington, Aqua~Media International, 2012, on CD,
50. BLAHA, P, – HORSKY O, Engineering-geological survey of the reservoir areas of dams, In: *Proceedings of HYDRO 2013 – Water Storage and Hydropower Development for Africa*, Addis Ababa, No 12,11, 2013, p, 1-8,
51. BROŽA, V, et al, *Přehrady (Dams)*, Praha: SNTL/ALFA, 1987,
52. BROŽA, V, et al, *Přehrady Čech, Moravy a Slezska (Dams of Bohemia, Moravia and Silesia)*, Knihy 555, 2005,
53. BŮŽKOVÁ, H, et al, *Metodika inženýrsko-geologického mapování zátopných oblastí přehrad v měřítku 1 : 5 000 – 10 000 (Methodology of engineering-geological mapping of backwater areas of dams on a scale of 1:5 000 – 10 000)*, OFTR, Brno: Geotest, 1964,
54. CASAJUS, J, et al, *Fases de la investigatión geotécnica en la concepción de Presas*, Buenos Aires: Actas Asociación Argentina, Vol, II, 1982,
55. CLAYTON, C, R, I, – SIMONS, N, E, – MATTHEWS M, C, *Site Investigation*, Granada Publishing, 1982,
56. ČABALOVÁ, D, – ŠAMALÍKOVÁ, M, *Inžinierska geológia (Engineering geology)*, Bratislava: Alfa, 1992,
57. DEERE D, U, Technical description of rock cores for engineering purposes, In *Felsmechanik und Ingenieurgeologie*, 1963, Vol, 1, No 1, p, 16-22,
58. DEERE, D, U, – MILLER, R, F, *Engineering Classification and Index Properties for Intact Rocks*, Technical Report AWFL-TR-65-116, Air Force Weapons laboratory, New Mexico: Kirtland, 1966, s, 308,
59. DOLEŽALOVÁ, M, – DROZD, K, Site investigations and calculations for two underground caverns in Peru, In *Proc, of 4 th Congress ISRM Montreux*, Vol, 2, 1979, p, 105-112, Sborník ISRM,
60. DRÁPAL, J, – BLÁHA, P, – HARTLOVÁ, L, *Zavedení a osvojení karotážní aparatury K500/2 (Implementation and acquisition of logging apparatus K500/2), Závěrečná zpráva o PÚTR 70,0182*, Brno: Geotest, 1980, MS,

61. DROZD, K, Variations in the Shear Strength of Rock Mass depending on the Displacement of the Test Blocks, In *Proceedings of the Geotechnical conference*, Oslo, 1967, p, 265-269,
62. DROZD, K, Field loading tests on the Vrchlice dam site, In *International symposium on rock mechanics related to dam foundations*, Brazil, 1978,
63. DROZD, K, *Mezinárodní doporučení pro posuzování diskontinuit v horninovém masivu (International recommendations for assessment of discontinuities in the rock mass)*, Geologický průzkum, 1980, 10, p, 312 – 314,
64. DROZD, K, O přetvárných charakteristikách zemin a skalních hornin, *Geotechnika*, 2/2001, p, 3-6,
65. DROZD, K,, LOUMA, B, The correlation of moduli of elasticity determined by microseismic measurement and static loading test, In *Procedings of the Congress of the ISRM*, Lisboa, 1967,
66. DVOŘÁK, A, *Základy inženýrské seismiky (Basics of engineering seismics)*, Skripta PS, Praha: PřFUK, 1969,
67. DZIEWANSKI, J, et al, *Badania geologiczne masiwów skalnych w budownictve wodnym*, Warszava: Wydawnictva Geologiczne, 1983,
68. Encyklopedický slovník geologických věd (ed, Svoboda J,), Academia, Praha, 1983, 851 s,
69. ESCOBAR, D, G, *Manual de geología para Ingenieros*, Manizales: Universidad Nacional de Colombia, 1998,
70. FEDORENKO, V, S, – DENIKAEV, Š, Š, – LIM, V, V, *Osnovnyje inženerno-geologičeskije aspekty problémy Sarezkogo ozera*, Nauka, Inženernaje geologija, 1981,
71. FOUSEK, J, et al, *Průzkum přehradních profilů geofyzikálními metodami (Survey of dam profiles by geophysical methods)*, Praha: Inženýrské Stavby, 24, 3, 1975, p, 138 – 143,
72. FOUSEK, J, et al, *Zásady IG průzkumu pro malé a střední údolní přehrady (Rules of EG survey for small- and medium-sized valley dams)*, Brno: Geotest, 1982, MS,
73. FRATTA D, – AGUETTANT J, *Introduction to Soil Mechanics Laboratory Testing*, Lynne Roussel-Smith, Baton Rouge, Louisiana, 2007, ISBN: 9781420045628,
74. FREJKOVÁ, L, *Příspěvek k poznání moravskoslezských sopek Nízkého Jeseníku (A contribution to the knowledge of Moravian-Silesian volcanos of the Nízký Jeseník Hills)*, Ostrava: Přírodovědecký sborník Ostravského kraje, roč, XIII, 3 – 4, 1952, p, 315 – 334,
75. FUSSGANGER, E, 2006 – oral communication
76. GARCÍA YAGUE, A, – ALVAREZ, G, Grandes deslizamientos Españoles, In *II, Simposio sobre taludes y laderas inestables*, Andorra la Vella, 1988, p, 599 – 612,
77. GARCÍA YAGÜE, A, *Temas de Geología aplicada a las Obras Públicas*, Madrid: Colegio de I,C,C, y P, 2000,
78. GELDART L,P,, SHERIFF R,E,, TELFORD W,M,: *Applied Geophysics – Second Edition*, Press Syndicate of the University of Cambridge, Melbourne, 1990,
79. GOLZÉ, R, A, *Hand book of Dam Engineering*, Van Nostrad Reinhold Company, New York, 1977,
80. GOODMAN, R, E, *Engineering Geology*, John Wiley & Sons, 1993,
81. GORJAJNOV, N, N, – LJACHOVICKIJ, F, M, *Seismičeskije metody v inženernoj geologii*, Moskva: Nedra, 1979,
82. GREČIŠČEV, E, K, *Metod rasčeta širiny zony razmyva běregov Bratskogo vodochranilišča*, Irkutsk: Akaděmija nauk SSSR, Sibirskoje otdělenije, 1961,
83. GROMA, B,: Metodika laboratórnych skúšok z mechaniky skalných hornín (Methodology of laboratory tests from mecjhanics of solid rocks), Rad pracovných pomôcok geologického útvaru, Žilina: IGHP, 1968,
84. HANÁK, Z, – MEJZLÍK, L, Zkušenosti s projektovou přípravou vodního díla Dalešice ve variantě s klenbovou přehradou hlavní nádrže (Experience with design preparation of the Dalešice water structure in the option of the arch dam of the main reservoir), In *O výstavbě klenbových přehrad*, Praha: ČSVTS, 1972, p, 195 – 212,
85. HAŠEK, V, – VESELÝ, I, – WOZNICA, L, *Uplatnění povrchové geofyziky při inženýrskogeologickém průzkumu přehradních profilů (Application of surface geophysics in engineering-geological survey of dam profiles)*, Praha: Sborník geologických věd, řada HIG, 1981, 15, p, 83 – 126,

86. HORÁK, L, – PASEKA, A, – POSPÍŠIL, P, Polní zkoušky a měření mechaniky hornin (stanovení in situ) (Field tests and measurement of rock mechanics (determination in situ), In *http://geotech,fce,vutbr,cz/studium/mech_hornin/mhig_11,pdf*, 2005,
87. HORSKÝ, O, *Průsaky z přehrady Foix ve Španělsku (Seepage from the Foix dam in Spain)*, Praha: Vodní hospodářství, 7, 1969, p, 201 –203,
88. HORSKÝ, O, *Zhodnocení inženýrskogeologických podmínek výstavby sypané zemní hráze na řece Jihlavě u Dalešic (Evaluation of engineering-geological conditions of the construction of an embankment earth-fill dam on the river Jihlava by Dalešice)*, Brno: Geotest, 1970, MS,
89. HORSKÝ, O, *Použití letecké fotogrammetrie pro sledování břehových změn v zátopových oblastech přehrad (Use of aerial photogrammetry for monitoring banks changes in backwater areas of dams)*, Praha: Geologický průzkum, 1971, 10, p, 300 – 302,
90. HORSKÝ, O, *Břehové změny v zátopové oblasti Oravské nádrže (Banks changes in the backwater area of the Oravice reservoir)*, Výzkumný úkol 72-19-0744, Brno: Geotest, 1974,
91. HORSKÝ, O, *Metodika a aplikace moderních metod při inženýrskogeologickém průzkumu pro přehradu v Dalešicích (Methodology and application of modern methods in engineering-geological survey for a dam at Dalešice)*, Kandidátská disertační práce (candidate dissertation thesis), Ostrava: VŠB, 1974,
92. HORSKÝ, O, *Sanace sesuvů na Oravské přehradě (Remediation of landslides at the Orava reservoir)*, Praha: Geologický průzkum, 1975, 2, p, 43 – 45,
93. HORSKÝ, O, *Investigaciones Ingeniero-Geológicas para las Obras Hidrotécnicas*, I, and II, Parte, Havana: MICONS, 1981,
94. HORSKÝ, O, Indice de calidad de la roca R,Q,D – uno de los parámetros básicos para clasificación de las rocas, In *IX, Jornada Científica,* Havana: Academia de Ciencias de Cuba, 1982, p, 189 – 192,
95. HORSKÝ, O, *Bases metodológicas para el desarrollo de las Investigaciones Ingeniero-Geológicas para las Obras Hidrotécnicas*, Havana: Voluntad Hidráulica, XX, N^0 62, 1983, s, 57 – 63,
96. HORSKÝ, O, *Investigaciones Ingeniero Geológicas para las Presas*, Praha: Český geologický úřad, 1984, s, 1 – 228,
97. HORSKÝ, O, *30 let Oravské vodní nádrže a její vliv na deformace břehů (30 years of the Orava water reservoir and its impact on banks deformation)*, Bratislava: Inženýrské stavby, 10, 1984, p, 489 – 498,
98. HORSKÝ, O, Využití RQD pro inženýrskogeologické hodnocení horninového prostředí (Application of RQD for engineering-geological evaluation of the rock environment), In *Inžiniersko-geologické sympózium*, Bratislava, 1984, p, 222 – 234,
99. HORSKÝ, O, *Inženýrskogeologický průzkum pro přehrady (Engineering-geological survey for dams)*, Edice GEODA, Praha: Geofond, 1990,
100. HORSKÝ, O, Stanovení korelačních vztahů mezi dynamickými moduly pružnosti a deformačními moduly v základech horní hráze přečerpávací elektrárny Centro Cuba (Determination of correlations between dynamic moduli of elasticity and moduli of deformation in the foundations of the upper dam of the Centro Cuba pumped storage hydroelectric plant), In *Geotechnické problémy energetickej výstavby*, Vysoké Tatry: ČSVTS, 1990, p, 41 – 44,
101. HORSKÝ, O, The Causes of Morphological Changes at the Water Edges of Orava - Reservoir in Slovakia, In *6th International Congress IAEG*, Rotterdam: Balkema, 1990, p, 2859 – 2868,
102. HORSKÝ, O, Diskuse ke stanovení rozsahu inženýrskogeologického průzkumu pro přehrady (Discussions on the determination of the extent of engineering-geological survey for dams), In *Metodické inovace v inženýrské geologii*, Příbram: Komitét sympozia HPVT, 1990,
103. HORSKÝ, O, Geotechnical research for the pumping storage station, In *7th International Congress IAEG*, Rotterdam: Balkema, 1994, p, 3919 – 3926,
104. HORSKÝ, O, – MÜLLER, K, – TRÁVNÍČEK, L, Průzkum porušení čedičového příkrovu v přehradním místě Slezská Harta geologicko-geofyzikálními metodami (Survey of fracturing of the basalt sheet at the Slezská Harta site by geological-gephysical methods), Praha: *Sborník geologických věd, řada HIG*, 1972, 10, p, 39 – 58,
105. HORSKÝ, O, – MÜLLER, K, *Sesuvy na březích Oravské přehrady (Landslides on the banks of the Orava reservoir)*, Praha: Sborník geologických věd, řada HIG, 1972, 10, p, 59 – 71,
106. HORSKÝ, O, – NOVOSAD, S, – MÜLLER, K, Zkušenosti z inženýrskogeologického průzkumu pro klenbovou hráz v Dalešicích (Experience from enginee-

ring-geological survey for an arch dam at Dalešice), In *Symposium o výstavbě klenbových přehrad*, Praha: ČSVTS, 1972, p, 154 – 177,
107. HORSKÝ, O, – WOZNICA, L, Problematika prognózy břehových změn v režimu kolísání hladiny údolních nádrží PVE (The issues of prognosis of banks changes in the regime of the level fluctuation of dam reservoirs of PSHEPs), In *Přehradní dny*, Ostrava, 1973, p, 140 – 147,
108. HORSKÝ, O, – JANDA M, *Údolní nádrž na Moravici u Slezské Harty (A dam reservoir on the river Moravice by Slezská Harta)*, Praha: Vodní hospodářství, 9, 1973, p, 239 – 243,
109. HORSKÝ, O, – MÜLLER, K, Trávníček L, Complex Documentation of exploratory workings, In *International Congress IAEG*, Sao Paulo, VII, 1974, p, 8,1 - 8,8,
110. HORSKÝ, O, – MÜLLER, K, *Inženýrskogeologické poměry přehradního místa Dalešice na řece Jihlavě Jihlavě (Engineering-geological conditions of the Dalešice dam site on the river Jihlava)*, Praha: Sborník geologických věd, řada HIG, 1974, 11, p, 125 – 160,
111. HORSKY, O, – MÜLLER, K, Rock environment – determining factor for projecting the hydrotecnical construction works, In *3rd International Congress IAEG*, sec, III, vol, 1, p, 143 – 151, Madrid, 1978,
112. HORSKY, O, – REYES, V, *Informe sobre las condiciones ingeniero-geológicas para la fundamentación del Projecto Preliminar del cierre del Conjunto Hidráulico Corojo*, Holguin: MICONS/ENIA, 1982,
113. HORSKY, O, – CONDE, M, *Las investigaciones geofísicas en el estudio ingeniero-geológica de obras hidrotécnicas*, La Habana: Revista Voluntad Hidráulica, 61, XX, 1983, s, 14 – 18,
114. HORSKÝ, O, – BLÁHA, P, Investigation of the disturbance of the basalt sheet at the dam site Slezská Harta using geological and geophysical methods, In *27, Geological Congress,* Moskva: MGK, Nauka, 1984, p, 57,
115. HORSKÝ, O, – SIMEONOVA, G, – SPANILÁ, T, *Vliv exogenních procesů na přetváření břehů vodních nádrží (Effect of exogeneous processes on the banks transformation of water reservoirs)*, Praha: Geologický průzkum, 1984, 6, p, 163 – 166,
116. HORSKÝ, O, – LINCER, L, – NEŠVARA, J, Ecological repairs of the Orava Dam reservoir shore banks, In *7th International Congress IAEG*, Rotterdam: Balkema, 1994, p, 3729 – 3737,
117. HORSKÝ, O, – SPANILÁ, T, Remodeling of water reservoir banks by exogenous processes, In *8th International Congress IAEG*, Rotterdam: Balkema, 1997, p, 2771 – 2716,
118. HORSKÝ, O, – BLÁHA, P, Inženýrskogeologický průzkum pro přehrady, Repronis, Ostrava, 2009, p, 226,
119. HORSKÝ, O, – BLÁHA, P, The Application of Engineering Geology to Dam Construction, Repronis, Ostrava, 2011, p, 296,
120. HOUSKA, J, – MATĚJOVSKÝ, J, – POLÁK, V,: Metodiky zkoušek základních fyzikálních a mechanických vlastností hornin (Methodologies of tests of basic physical and mechanical properties of rocks), Praha: Hornický ústav ČSAV, 1963,
121. HRDÝ, J, Srovnání závěrů inženýrskogeologického průzkumu a geotechnického sledu obtokových štol na VD Dalešice (Comparison of conclusions of the engineering-geological survey and geotechnical sequence of diversion tunnels at the Dalešice WS), In *Přehradní dny*, Ostrava, 1973, p, 166 – 175,
122. HRDÝ, J, VD *Dalešice – zpráva o průzkumných pracích na pravobřežním svahu hlavní základové jámy (Dalešice WS – report on survey work on the right,bank slope of the foundation pit)*, Geotest, Brno, 1973, MS,
123. HRDÝ, J, et al, *Závěrečná zpráva o předběžném inženýrsko-geologickém průzkumu pro Cha Centro Cuba (Final report on preliminary engineering-geological survey for Cha Centro Cuba)*, Brno: Geotest, 1981, MS,
124. HRDÝ, J, – VALEŠ, V, *Morfostrukturní analýza při inženýrskogeologickém a hydrogeologickém mapování (Morphostructural analysis in engineering-geological and hydrogeological mapping)*, Praha: Geologický průzkum, 1983, 6, p, 166 – 167,
125. HRUBEC, K, *Zářez dálnice u města Teruel (Španělsko)*, G IMPULS, Praha, 1999, MS,

126. HUNT R, E, *Geotechnical Investigation Methods: A Field Guide for Geotechnical Engineers* Practicing Geotechnical Engineer, Bricktown, New Jersey, 2006, ISBN: 9781420042740,
127. *International Congress on Large Dams* (XII), Mexiko, 1976,
128. *Itaipu, Principal Technical Features*, Itaipu Binational, Foz de Iguacu, 29 s,
129. *Inventario de Presas Espaňolas*, ICOLD 1973, Proceedings of the International Congress on Large Dams, Mexico, 1976,
130. JANDORA, J, – ŘÍHA, J, *Porušení sypaných hrází v důsledku přelití (Failure of embankment dams due to spillover)*, Práce a studie, Brno: Ústav vodních staveb, FAST VUT in Brno, 2002, 188 p, ISBN 80-86433-15-5,
131. KACHUGIN, E, G, *Někotorye zakonomernosti procesov pererabotky beregov vodochranilišč*, Voporosy ustojčivosti sklonov 35, Moscow: ANSSSR, 1961,
132. KALÁB, Z, *Seizmická měření v geotechnice (Seismic measurements in geotechnics)*, Ostrava: VŠB – TU, FS, 2008,
133. KAROUS, M, *Geoelektrické metody průzkumu (Geoelectrical methods of survey)*, Praha: SNTL, 1989,
134. KAROUS, M, *Geofyzikální metody v inženýrské geologii a geotechnice (Geophysical methods in engineering geology and geotechnics)*, Praha: Geonika s,r,o, 1998, special publication, MS,
135. KARPYSHEV, E, S, *Inženiernogeologičeskie izyskanija dlja strojitělstva gidrotěchničeskich sooruženij*, Moscow: Eněrgija, 1980,
136. KAZDA, I, – BROŽA, V, *Teorie přehrad (Theory of dams)*, Praha: ČVUT, 1971,
137. KEIL, K, *Ingeniergeologie und Geotechnik*, VEB Wilhelm Knapp Verlag, Halle, 1954,
138. KENNETH D, W,, – DONALD A, B, *Dam Foundation Grouting*, American Society of Civil Engineers, ASCE PRESS, Reston, Virginia, 2007,
139. KHMELEVSKOY, V, K, *Osnovnoj kurz elektrorazvedki*, Izdatelstvo MGU, Moskva, 1975,
140. KELLY, W, E, – Mareš S, (Eds), *Applied geophysics in hydrogeological and engineering practice*, Elsevier, Amsterodam, 1993,
141. KOBR, M, et al, *Petrofyzika*, Charles University textbook, Praha: Karolinum, 1997,
142. KOPECKÝ, M,: Influence of extreme climatic conditions upon lanslides development in the Slovak Republic, In *Slovak Geological Magazine*, Volume 12, No, 1, Bratislava: State Geological Institute of Dionýz Štúr, 2006, p, 63-68, ISSN 1335-096X,
143. KOPECKÝ, M, *Svahové pohyby a ich vplyv na prípravu a prevádzku vodných stavieb*, habilitačná práca, Slovenská technická univerzita v Bratislave, Stavebná fakulta, Katedra geotechniky, 2009, p, 1-124, MS,
144. KOPECKÝ, M, – MARTINČEKOVÁ, T, ŠIMEKOVÁ, J, ONDRÁŠIK, M, Atlas zosuvov – výsledky riešenia geologickej úlohy, Landslide atlas – results of the geological project, I*n: Geológia životné prostredie*, Bratislava: Štátny geologický ústav Dionýza Štúra, 2008, p, 105-110, ISBN 978-80-89343-06-5,
145. KOŘALKA S, *Závěrečná zpráva o karotáži při inženýrskogeologickém průzkumu prodloužení trasy A metra v Praze*, Praha: Aquatest, 2008, MS,
146. KOŘALKA S, – PROCHÁZKA M, *Zpráva o karotážním měření na VD Guiamets (Report on logging measurement at the Guiamets WS)*, Madrid: Geoinza, 1996, MS,
147. KOŘÍNEK, R, – ALDORF, J, *Geotechnický monitoring* (Geotechnical monitoring), university textbook, Ostrava: VŠB, 1994,
148. KOS, J, – ZAJÍC, J, *Technická geologie (Technical geology)*, Praha: SNTL, 1961,
149. KRATOCHVIL, S, *Údolné přehrady (River reservoirs)*, Slovak Academy of Sciences, Bratislava 1953,
150. KRÁSNÝ, J, – SHARP, J, M,, eds,, *Groundwater in fractured rocks*, IAH Selected Papers 9, Taylor and Francis, 2007, 648 s,
151. KRÁSNÝ, J, et al, *Regionální hydrogeologie prostých minerálních vod*, Česká geologická služba, Praha, in print,
152. KRČMA, L, In HANÁK, Z, MEJZLÍK, L, *Vodní dílo Dalešice (Dalešice water structure)*, Praha: HDP, 1975,
153. LEE, K, L, AND DUNCAN, J, M, *Landslide of April 25, 1974, on the Mantaro River, Peru*, Nat'l, Acad, Sciences, Wash,, D,C,, 1975, 72 p,
154. LIM, V, V, – AKDODOV, J, *Opolzni Sareza*, Trudy Tadžikglavgeologii, Tashkent: Uzbekgeologija, 1998,

155. LINK, H, *Uber die Unterschiede statisch, dynamisch und seismisch ermittelten Elaszizitatsmodulen von Gestein und Gebirge*, Wien: Geologie und Bauwesen, Jg, 27, H,3-4, 1962,
156. LJACHOVICKIJ, P, M, – CHMELEVSKOJ, V, K, – JAŠČENKO, Z, G, *Inženernaja geofyzika*, Moskva: Nedra, 1989, p, 1 – 145,
157. LOVRIE, W, *Fundamentals of Geophysics*, Second edition, Cambridge: University Press, 2007,
158. LUGEON, M, *Barrages et Géologie*, Lausanne, 1933,
159. LUKÁČ, M, – BEDNÁROVÁ, E, Navrhovanie a prevádzka vodných stavieb (Design and operation of water structures), In: *Sypané priehrady a hrádze*, Bratislava: Jaga group, 2006, p, 183, ISBN 80-8076-015-2,
160. LUKEŠ, J, *Zpráva o karotážním měření ve vrtech JM1014, JM1024, JM1034, JM1044 (Report on logging measurement in boreholes JM1014, JM1024, JM1034, and JM1044)*, Praha: Aquatest, 2002, MS,
161. LUKEŠ, J, *Zpráva o karotážním měření ve vrtech D1, IV4 a PIM44 (Report on logging measurement in boreholes D1, IV4 and PIM44)*, Praha: Aquatest, 2003, MS,
162. LUKEŠ, J, *Závěrečná zpráva o karotážním měření ve vrtech MEL1 až MEL6 (Final report on logging measurement in boreholes MEL1 to MEL6)*, Praha: Aquatest, 2006, MS,
163. LUKEŠ, J, – Pitrák M, Závěrečná zpráva o karotážním měření ve vrtech KV-1, PDV-1, PZV-1, CTV-1 *(Final report on logging measurement in boreholes KV-1, PDV-1, PZV-1, CTV-1)*, Praha: Aquatest, 2010, MS,
164. MADURGA, L, – DOMÉNECH, J, Study of Seepage from a Reservoir Situated on Calcareous Land, In *IX, Int, Congress for Large Dams*, Istanbul, 1957,
165. MALGOT, J, – KLEPSATEL, F, – TRÁVNÍČEK, I, *Mechanika hornín a inžinierska geológia (Rock mechanics and engineering geology)*, Bratislava: Alfa, 1992,
166. MAREŠ, S, et al, *Úvod do užité geofyziky (Introduction to applied geophysics)*, Praha: SNTL-Alfa, 1979,
167. MAREŠ, S, et al, *Geofyzikální metody v hydrogeologii a inženýrské geologii (Geophysical methods in hydrogeology and engineering geology)*, Praha: SNTL, 1983,
168. MARSCHALKO, M, Multimediální výukové texty inženýrské geologie (Multimedia instructional texts of engineering geology), p, 19 – 21, *Pražské geotechnické dny 2003*, p, 22 – 23,
169. MASSARCH, K, R, *Geophysical Methods for Geotechnical, Geoenvironment and Geodynamic Site Characterization*, Melbourne: Contribution on 3rd IWAGRSE, 2000,
170. MATULA, M, *Inžinierskogeologické štúdium horninového prostredia a geodynamických procesov (Engineering-geological study of the rock environmentand geodynamic processes)*, Bratislava: Vydavateľstvo Slovenskej akadémie vied (Publishing House of the Slovak Academy of Sciences), 1979,
171. MATULA, M, Rock and soil description and classification for engineering geological mapping, In *Bulletin of the International Association of Engineering Geology*, Aachen: N^0 24, 1981, p, 235 – 274,
172. MATULA, M, – PAŠEK, J, *Zásady inženýrsko-geologického mapování (Rules of engineering-geological mapping)*, Praha: Sborník geologických věd, HIG series, 1966, 5, p, 161 – 174,
173. MATULA, M, – HRAŠNA, M, *Typologická rajonizácia v inžinierskej geológii (Typological zoning in engineering geology)*, Acta geologica et geographica Universitatis Comeniane, Bratislava: Geologica, 29, 1976,
174. MATULA, M, et al, *Moderné metódy hodnotenia horninového a životného prostredia (Modern methods of evaluation of the rock and living environment)*, Bratislava: Univerzita Komenského, 1988,
175. MENCL, V, *Mechanika zemin a skalních hornin (Mechanics of soil and solid rocks)*, Praha: Academia, 1966,

176. MICHLÍČEK, E, *Hydroekologické mapy pro potřeby správních a vodohospodářských orgánů (Hydroecological maps for the needs of administrative and water-management authorities)*, EKO ekologie a společnost, Praha: ČNTZ, VIII, 4, 1997,
177. MÍSAŘ, Z, *Regionální geologie světa*, Akademia, Praha, 1987,705 s,
178. MJULER, K, – NOVOSAD, S, Metodika i ocenka elastičnych svojstv gornych porod na plotinnom profile Slezská Garta, In *Proceedings of XXXI IGS*, I, Gdaňsk, 1986, p, 29 – 37,
179. MRLINA, J, Březové Hory – mikrogravimetrický průzkum podél silnice I/18 a dalších rizikových místech (Březové Hory – microgravimetrical survey along I/18 Road and other risk places), In BLÁHA, P, – DURAS, R, *Březové Hory, doplňková zpráva o geofyzikálním měření v okolí silnice I/18*, Brno: Geotest, 1998, MS,
180. MÜLLER, K, Geophysical methods in prospecting sites for engineering works projected, In *II, International Conference: Application of geophysics in HG and EG*, Brno, 1976,
181. MÜLLER, K, *Metodologie inženýrskogeofyzikálního výzkumu horninového masivu (Methodology of engineering-geological research of the rock mass)*, Doktorská dizertační práce (doctoral dissertation thesis), Ostrava: VŠB, 1987,
182. MÜLLER, K, et al, Zpráva o geofyzikálním průzkumu přehradního místa ve Slezské Hartě, okres Bruntál (Report on a geophysical survey of a dam site at Slezská Harta, Bruntál District), In HORSKÝ, O, *Závěrečná zpráva o inženýrsko-geologickém průzkumu přehradního profilu a zátopného území přehrady na řece Moravici ve Slezské Hartě*, II, etapa, Brno: Geotest, 1968, MS,
183. MÜLLER, K, – TRÁVNÍČEK, L, – BLÁHA, P, *Dalešice II – Mohelno, část 4, upřesnění průběhu tektonické poruchy v údolí a styku hadce a granulitu (Dalešice II – Mohelno*, part 4, specification of the course of a tectonic fracture in the valley and the serpeninite/granulite contact), Brno: Geotest, 1969, MS,
184. MÜLLER, K, et al, Dalešice I – sypaná hráz, Zpráva o doplňkovém geofyzikálním průzkumu (Dalešice I – embankment dam, Report on an additional geophysical survey), In HORSKÝ, O, *Zhodnocení inženýrskogeologických podmínek výstavby sypané zemní hráze na řece Jihlavě u Dalešic*, Brno: Geotest, 1970, MS,
185. MÜLLER, K, et al, *Zpráva o geofyzikálním průzkumu přehradního místa na řece Kamenici v Josefově Dole u Liberce (Report on a geophysical survey of a dam site on the river Kamenice at Josefův Důl by Liberec)*, Ostrava: VŠB, 1970, MS,
186. MÜLLER, K, et al, *Zpráva o geofyzikálním průzkumu v oblasti hydrocentrály na VD Dalešice (Report on a geophysical survey in the area of a powerhouse at the Dalešice WS)*, Ostrava: VŠB, 1971, MS,
187. MÜLLER, K, – BLÁHA, P, Některé výsledky geofyzikálního sledování pohybu podzemní vody (Some results of the geophysical monitoring of groundwater movement), In *O využití geofyziky v IGHG*, Brno, 1972, p, 149–167,
188. MÜLLER, K, et al, Zkušenosti z použití geofyzikálních metod při inženýrskogeologickém průzkumu přehradních míst (Experience from the use of geophysical methods in engineering-geological surveys of dam sites), In *Aplikace geofyziky v IGHG*, I, díl (part I), Brno, 1972, p, 169 – 180,
189. MÜLLER, K, et al, Komplexní dokumentace průzkumných děl (Comprehensive documentation of exploratory workings), In *Přehradní dny 1973*, Ostrava, 1973, p, 100 – 111,
190. MÜLLER, K, – MÜLLEROVÁ J, Metodika orientačního posouzení geotechnických vlastností žulového masivu v přehradním místě Josefův Důl (methodology of the orientational assessment of geotechnical properties of the granite massif at the Josefův Důl dam site), In *Přehradní dny*, Hradec Králové, 1975, p, 64 to 74,
191. MÜLLER, K, – BLÁHA, P, – NEŠVARA, J, Geofyzikální dokumentace průzkumných štol (Geophysical documentation of exploratory workings), In *Geotechnický prieskum pre tunely a navrhovanie tunelových ostení*, ČSVTS Vysoké Tatry, Košice: Dům techniky, 1978, p, 211–219,
192. MUZIKÁŘ, R, Korelační analýza kolísání hladiny ve vrtech (Correlation analysis of water level fluctuation in boreholes), In HORSKÝ, *O, Inženýrsko-geologický průzkum přehradního místa Slezská Harta, 1, ucelená část (integrated part)*, Brno: Geotest, 1984, MS,

193. NEMČOK, A, – PAŠEK, J, – RYBÁŘ, J, *Dělení svahových pohybů (Division of slope movements)*, Praha: Sborník geologických věd, HIG series, 1974, 11, p, 77 – 97,
194. NEŠVARA, J, – BLÁHA, P, *Zpráva o inženýrskogeologickém průzkumu některých dílčích objektů PVE Hrhov (Report on an engineering-geological survey of some partial objects of the Hrhov PSHEP)*, Brno: Geotest, 1977, MS,
195. NEŠVARA, J, – BLÁHA, P, – MÜLLER, K, *Zpráva o inženýrskogeologickém průzkumu pro PVE Malá Vieska – II, etapa (Report on an engineering-geological survey for the Malá Vieska PSHEP – stage II)*, Brno: Geotest, 1977, MS,
196. NEŠVARA, J, – BLÁHA, P, *Některé problémy monitorování sesuvů (Some problems of landslide monitoring)*, Edice Geoda, vol, 4, Praha/Brno: Geofond ČR/Geotest, 1991,
197. NIKITIN, V, N, *Osnovy inženernoj seismiki*, Moskva: Izdatelstvo Moskevskogo universiteta, 1981,
198. NIYAZOV, R, A, *Formirovanije krupnych opolznej srednej Asii*, Tashkent: Fan, 1982,
199. NIYAZOV, R, A, *Sovremennyje metody izměrenija napravljenija porovo davlenija i dviženij opolzňa na glubině*, Tashkent: Fan, 1989,
200. NIYAZOV, R, A, Opolzni Uzbekistana, Fan, Tashkent, 2009,
201. NIYAZOV, R, A, 2012 – oral communication,
202. NOVOSAD, S, *Šance – sesuvy, Závěrečná zpráva o inženýrskogeologickém průzkumu (Šance – landslides, Final report on an engineering-geological survey)*, Brno: Geotest, 1967, MS,
203. NOVOSAD, S, Establishing conditions of equilibrium of landslides in dam reservoirs by means of geoacoustic (rock noise) method, In *Bull, Int'l, Assoc, Engrg, Geology*, No, 20, p, 138-144, 1979,
204. NOVOSAD, S, Key-role of monitoring landslide for risk management, In *The 1st European Conference on Landslides*, Praha, 2002,
205. NOVOSAD, S, – RYBÁŘ, J, *Nepropustnost levého břehu nádrže u Žermanic*, Praha: Věstník ÚÚG, 23, 2, 1958, p, 106 – 119,
206. NOVOSAD, S, – HORSKÝ, O, *Inženýrskogeologické podmínky výstavby přehrad na Ostravsku (Engineering-geological conditions of dam construction in the Ostrava Region)*, Praha: Vodní hospodářství, 1973, p, 219 – 224,
207. NOVOSAD, S, – BARVÍNEK, R, TORRE, M,S, Estudio de estabilidad del Derrumbe No 5, En el Reservorio de Tablachaca de la Central Hidroelectrica del Mantaro, In *The 6th Panamericen Congress of Soil Mechanics and Foundation Engineeging*, Lima, Vol, 1, 1979, p, 331-334,
208. NOVOSAD, S, – WOZNICA, L, *Prognóza změn břehového pásma v okolí vodních nádrží ČSSR z inženýrskogeologického hlediska (Prognosis of changes of the banks zone in the vicinity of water reservoirs of the CSSR)*, Brno: Geotest, 1987, MS,
209. OKAMOTO S,, Present trend of earthquake-resistant design of large dams, In *The assesment and mitigation of earthquake risk*, UNESCO 1980, ISBN 92-3-3011451,
210. ONDRÁŠIK, R, – RYBÁŘ, J, *Dynamická inžinierska geológia (Dynamic engineering geology)*, Bratislava: Slovenské pedagogické nakladateľstvo, 1991,
211. ONDRÁŠIK, R, – VLČKO, J, – FENDEKOVÁ, M, *Geologické hazardy a ich prevencia (Geohazards and their prevention)*, Univerzita Komenského Bratislava ve vydavatelství UK, 2011, ISBN 978_80-223-2956-9, p, 286,
212. PANTL, V, *Zpráva o pokusném ultrazvukovém karotážním měření v Dalešicích (Report on experimental ultrasonic logging measurement)*, Brno: ÚGF, 1969, MS,
213. PAŠEK, J, – MATULA, M, et al, *Inženýrská geologie I, II (Engineering geology I and II)*, Praha: ČMT – TP 76, 1995,
214. PAVLÍK, J, *Geotechnické způsoby určování stability skalních stěn (Geotechnical methods of determination of stability of rock walls)*, Praha: SNTL, 1981,
215. PAVLÍK, J, *Zpráva o geotechnickém průzkumu horninového masivu pro přečerpávací vodní elektrárnu Hrhov (Report on a geotechnical survey of the rock mass for the Hrhov pumped storage hydroelectrical plant)*, Brno: Geotest, 1975, MS,
216. PAZDÍREK, O, et al, *Quo vadis, DC Resistivity? New Ways in Direct Current Resistivity Field Acquisition Technology*, Praha: EGRSE, 2, p, 5 – 10, 1997,

217. PERTOLDOVÁ, J, *Dalešice – seismické měření (Dalešice – seismic measurement)*, Praha: Stavení geologie, 1968, MS,
218. PETER, P, – VOTRUBA, L, – MEJZLÍK, L, *Údolné nádrže a priehrady (Dam lakes and reservoirs)*, Bratislava: Slovenské vydavatelství technické literatury, 1967,
219. PITRÁK, M, *Vývoj speciální sondy na detekci směrů horizontálního proudění podzemní vody (Development of a special probe for the detection of directions of the horizontal flow of groundwater)*, Praha: Aquatest, 2006, MS,
220. PITRÁK, M, et al, *Výzkum vlivu mezizrnné propustnosti granitů na bezpečnost hlubinného ukládání do geologických formací a vývoj metodiky a měřící aparatury (Research into the effect of intergranular permeability of granite on the safety of deep deposition into geological formations and the development of procedures and measuring apparatus)*, Aquatest, 2010, MS,
221. POLÁČEK A, 2000 – ústní sdělení (oral communication)
222. POLÁŠEK, S, *Propustnost podzákladí přehradních míst ve vztahu k napjatostním poměrům a deformacím horninového masivu (Permeability of the subsoil of dam sites in relation to the states of stress and deformations of the rock mass)*, Praha: Geologický průzkum, 1971, 4, p, 107 – 110,
223. POUBA, Z, *Geologické mapování (Geological mapping)*, Praha: ČSAV, 1959,
224. PRUŠKA, J, *Geomechanika, Mechanika hornin (Geomechanics, Soil mechanics)*, Praha: Vydavatelství ČVUT, 2002,
225. REBOLLO, A, L, Étude du terrain au barrage de Susqueda, In *IX, Mezinárodní přehradní kongres*, Istanbul, 1967,
226. *Recommendations on site investigations techniques*, ISRM, 1975,
227. *Revista de Obras Publicas*, Madrid: MOP, 1972,
228. REYES, V, – HORSKY O, – STOICHEV, E, *Informe sobre las Condiciones Ingeniero – geológicas e hidrogeológicas para la fundamentación del cierre del Conjunto Hidráulico „Charco Redondo“ sobre el rio Cautillo*, Holguin: MICONS, 1981, MS,
229. ROTH, V, – TKANÝ, Z, *Sondovací práce ve stavebnictví (Subsurface exploration work in civil engineering)*, Praha: SNTL, 1959,
230. ROZSYPAL, A, *Kontrolní sledování a rizika v geotechnice (Check monitoring and risks in geotechnics)*, Bratislava: Jaga group, 2001, ISBN 80-88905-44-3
231. RYBÁŘ, J, Landslides on the sides of the Nechranice reservoir, In *Guide of the Symposium Int, Assoc, Eng, Geol,*, Praha, 1977,
232. RZHEVSKIY, V, V, – NOVIK, G, J, *Osnovy fiziky gornych porod*, Moskva: Nedra, 1978,
233. řezníček p, Penetrační metody (Penetration Methods), PP presentation, Geotest, Brno, 2007, MS,
234. SAENZ, C, et al, *Ámbito geológico de la presa*, Madrid: Separata de la obra Grandes Presas, 1974,
235. SAVICH, A, I, *Seismoakustičeskije metody izučenija masivov skalnych porod*, Moskva: Nedra, 1969,
236. SAVICH, A, I, et al, Seismic Survey of the Ingouri arch dam pit, In *Third Congress of Rock Mechanics*, Vol, II, Part B, Denver, 1974,
237. SELLI, R, TREVISAN, L, *Carrateri e interpretazione della frana del Vaiont*, G, Geol, XXXII, 1964, s, 1 – 52,
238. SERAFIM, J, L, – PEREIRA, J, P, Consideration of the geomechanics classification of Bieniawski, Proc, *Int, Symp, on Engineering Geology and Underground constructions*, pp, 1133 – 1144, 1983,
239. SCHUSTER R, L, Dams Built on Pre-existing Landslides, *Proceedings, GeoEng*, 2000, Melbourne, Nov, 19-24, v, 1, p, 1537-1589,
240. SCHUSTER, R,L, – SALCEDO, D,A, – VALENZUELA, L, An overview of catastrophic landslides of South America in the 20th Century, In *S, Evans and J, DeGraff, eds,, Catastrophic landslides of the 20th century*, Reviews in Eng, Geol,, Geol, Soc, of America, (in press)
241. SUSILO H, – Nugroho I, Sediment Transport at Bakaru Reservoir, South Silawesi, Indonesia, in: *Proceedings "Water Resources and Renewable Energy Development in Asia*, Danang, Vietnam, Hydropower&Dams, 2008,
242. SERAFIM, J, L, – PEREIRA, J, P, Consideration of the geomechanics classification of Bieniawski, In *Proc, Int, Symp, on Engineering Geology and Underground Constructions*, 1983, p, 1133 - 1144,
243. SPANILÁ, T, – HORSKÝ, O, – BANACH, M, Slides and sliding in the water reservoirs banks, In *Landslides*, Lisse: Swets & Zeitlinger, 2002, p, 315 – 319,

244. SPANILÁ, T, – SIMEONOVÁ, G, *Hodnocení vlivu technogenního faktoru na přetváření břehů vodních nádrží (Evaluation of the effect of the technogenic factor on the transformation of banks of water reservoirs)*, Praha: Vodní hospodářství, A series, 3/1989, p, 78 – 82,
245. SPANILÁ, T, *Hodnocení abraze v jílovitých horninách na březích vodního díla Nechranice (Evaluation of abrasion in clayey rocks on the banks of the Nechranice water reservoir)*, Praha: Ústav geologie a geotechniky ČSAV, 1981, MS,
246. STAGG, K, G, – ZIENKIEWICZ, O, C, (editoři): *Rock Mechanics in Engineering Practice*, John Wiley & Sons, 1969,
247. STARUDUBTSEV V, M, – BOGDANETS V, A, New delta formation in the large water reservoir, in: *Proceedings "Water Resources and Renewable Energy Development in Asia,* Chiang Mai, Thailand, Hydropower&Dams, 2012,
248. STARÝ V, *Geotechnický a radonový průzkum pro výstavbu velkoprodejny potravin na lokalitě Libochovice"*, GEOTREND, Slaný, 2010, MS,
249. STRAKA, J, Mechanika hornin (Rock mechanics), Praha: ČVUT, 1987,
250. SUSILO, H, – NUGROHO, I, Sediment transport at Bakaru reservoir, *In Proc, Water Recources and Renewable Energy Development in Asia*, Hydropower & Dams, Danang, 2008,
251. SYNEK, V, *Zpráva o geofyzikálním měření v prostoru naleziště kamene na lokalitě Dlouhá Loučka (Report on geophysical measurement in the area of a stone deposit at the site Dlouhá loučka)*, Brno: Geofyzika, 1977, MS,
252. ŠAMALÍKOVÁ, M, – PIVOVARČIOVÁ, J, – PROSTĚJOVSKÁ, M, Vrtný periskop BP – 34 v ČSSR (A borehole periscope BP – 34 in the CSSR), Praha: Geologický průzkum, 1974, 5, p, 152 – 153,
253. ŠAMALÍKOVÁ, M, – TRÁVNÍČEK, I, *Inženýrská geologie a mechanika hornin (Engineering geology and soil mechanics)*, Praha: SNTL, Praha, 1984,
254. ŠAMALÍKOVÁ, M, *Inženýrská geologie a hydrogeologie (Engineering geology and hydrogeology)*, Brno: Akademické nakladatelství CERM, 1996,
255. ŠKOPEK, J, – ŤAVODA, O, – DROZD, K, *Mechanika hornin I (Soil mechanics I)*, Praha: SPN, 1986,
256. TESAŘ, O, Review of Rock Classifications for Underground Structures and its Relations to Classification QTS (in Czech), Praha: *Inženýrské stavby*, 6, 1989, p, 261 – 265,
257. TESAŘ, O, The Design of Rock Classification for Underground Structures in Prague (in Czech), *Zpravodaj Metro*, 1, Praha, 1977,
258. TORRE, M,S, Estudio de estabilidad del Derrumbe No 5, En el Reservorio de Tablachaca de la Central Hidroelectrica del Mantaro, In *The 6th Panamericen Congress of Soil Mechanics and Foundation Engineeging*, Lima, 1979,, Vol, 1, p, 331-344,
259. TRÁVNÍČEK, I, – HRDÝ, J, Mechanika hornin (Rock mechanics), Praha: SNTL, 1977,
260. TRÁVNÍČEK, L, – ŠVEC, J, – BLÁHA, P, *Zpráva o výzkumném gamagamametrickém měření na lokalitě Dlouhé Stráně – Hladová chata (Report on research gamma-gammametric measurement at the site Dlouhé Stráně)*, Brno: Geotest, 1971, MS,
261. *Usina Hidreléctrica de Itaipu*, Itaipu, Binacional, Foz de Iguacu,135 s,
262. VALLEJO, G, L, *Ingeniería Geológica*, Madrid: Pearson Educación, 2002,
263. VALLEJO, G, L, *Geological Engineering*, Universidad Complutense de Madrid, Mercedes Ferrer, Instituto Geológico y Minero de España, Madrid, 2010, p, 725,
264. VALTR, V, – PANTL, V, *Zpráva o geofyzikálním měření ve vrtech v oblasti hydrocentrály VD Dalešice (Report on geophysical measurement in boreholes in the area of a powerhouse of the Daleěice WS))*, Brno: Geotest, 1971, MS,
265. VALTR, V, *Aplikace karotážních metod v inženýrské geologii (Application of logging methods in engineering geology)*, Výzkumná zpráva, Brno: Geofyzika, 1977, MS,
266. VANÍČEK, I, *Mechanika zemin*, (Soil mechanics) Praha: ES ČVUT, 1983,
267. VELEN, T, – TLUCZEK, R,, – BASSON, J,, – TROUILLE, B, Initial design optimization of the Lima pumped-storage scheme, The International Journal Hydropower and dams, Aqua Media International, Sutton, UK, Issue 6, 2008,

268. VERFEL, J, – TKANÝ, Z, *Těsnění základových půd (Sealing of subsoils)*, Praha: SNTL, 1974,
269. VERFEL, J, *Rock grouting and diaphragm wall construction*, Amsterdam, Oxford, New York, Tokyo: ELSEVIER, 1989,
270. VESELÝ, I, *Zpráva o I, etapě doplňujícího inženýrskogeologického průzkumu pro projektový úkol vodní nádrže na Oslavě u Dlouhé Loučky (Report on stage I of an additional engineering-geological survey for a project task of a water reservoir on the Oslava by Dlouhé Loučky)*, Brno: Geotest, 1977,
271. VLASTNÍK, M, – BLÁHA, P, Geofyzikální měření, In LINCER, L, *Předběžný inženýrskogeologický průzkum pro vodní nádrž Budišov nad Budišovkou (Preliminary engineering-geological survey for the Budišov nad Budišovkou water reservoir)*, Brno: Geotest, 1984, MS,
272. VLASTNÍK, M, Seismické prozařování mezi dvojicemi vrtů, In HORSKÝ, O, et al, *Závěrečná zpráva o inženýrskogeologickém průzkumu pro studii proveditelnosti Centro Cuba (Final report on an engineering-geological survey for a feasibility study of Centro Cuba)*, Brno: Geotest, 1987, MS,
273. VOROPINOV, J, et al, *Geomechanické profilování průzkumné štoly Št 14 a jádrových vrtů v prostoru přehrady v Dalešicích (Geomechanical profiling of exploratory gallery Št 14 and core boreholes in the area of a dam at Dalešice)*, Ostrava: VŠB, 1971, MS,
274. VOROPINOV, J, *Mechanické profilování v inženýrskogeologických průzkumných dílech (Mechanical profiling in engineering-geological exploratory workings)*, Žilina: Sborník SVTS, IGHP, 1971, p, 120 – 142,
275. VOTOČEK, R, – DOMANSKÝ, J, Aplikace mikroseismických metod v inženýrské geologii (Application of microseismic methods in engineering geology), In *Aplikace geofyziky v IGHG, Brno*, 1972, p, 357 – 362,
276. WALTHAM, A, C, *Foundations of Engineering Geology*, Blackie Academic & Professional, 1995,
277. WEST, T, R, *Geology Applied to Engineering*, New Jersey: Prentice Hall, 1995,
278. WOZNICA, L, *Inženýrskogeologické rajónování Karpatského flyše Moravy z hlediska výstavby přehrad (Engineering-geological zoning of the Carpathian Flysch of Moravia from the point of view of dam construction)*, Praha: ČVUT, 1965,
279. WOZNICA, L, *Inženýrskogeologické rajónování Karpatského flyše Moravy z hlediska výstavby přehrad (Engineering-geological zoning of the Carpathian Flysch of Moravia from the point of view of dam construction)*, Praha: Vodní hospodářství, 17, 8, 1967,
280. WOZNICA, L, Inženýrskogeologický průzkum v místě hráze (Engineering-geological survey at dam sites), In *Zakládání konstrukcí nižších zemních hrází*, Ostrava: Dům techniky, 1984,
281. WOZNICA, L, Inženýrskogeologický průzkum zátopy a zemníků (Engineering-geological survey of backwater areas and borrow pits), In *Zakládání konstrukcí nižších zemních hrází, II*, Ostrava: Dům techniky, 1985,
282. WOZNICA, L, Požadavky na inženýrskogeologický průzkum při výstavbě malých vodních nádrží (Demands for the engineering-geological survey in construction of small water reservoirs), In *Vybrané otázky o výstavbě a provozu malých vodních nádrží*, Brno: ČSSU, 1991,
283. WOZNICA, L, *Přetváření břehů zátopových oblastí přehrad (Transformation of banks of backwater areas)*, Brno: Geotest, 1967, MS,
284. YEROFEYEV, L, Y, et al, *Fizika gornych porod*, Tomsk: ITPU, 2006,
285. YESENOV U, Y, – DEGOVETS A, S, Catastrofic mudflows on the Bolshaya Almatinka River in 1977, in: Soviet Gydrogeology: Selected Paper, Vol, 18, No, 2, p, 158-160,
286. YESENOV U, Y, – DEGOVETS A, S, Protection of the city of Alma Ata from mud – rock flows, in: *Landslides and mudflows – Reports of Alma – Ata International Seminar*, A, Sheko ed,, Alma-Ata, 2008,
287. ZÁRUBA, Q, – MENCL, V, *Inženýrská geologie (Engineering geology)*, Praha: Academia, 1957, 1974,
288. ZÁRUBA, Q, – MENCL, V, *Sesuvy a zabezpečování svahů (Landslides and slope stabilisation)*, Praha: Academia, 1987,
289. ZÁRUBA, Q, – VACHTL, J, – POKORNÝ, M, *Základy geologie a petrografie pro stavební fakulty (Basics of geology and petrography for faculties of civil engineering)*, Praha: SNTL-ALFA, 1974,
290. ZAVORAL, J, et al, Metodiky laboratorních zkoušek mechaniky zemin a hornin (Methodologies of laboratory tests of soil and rock mechanics), Praha: ČGÚ, 1987,

291. ZEMAN, V, – MÜLLER, K, *Mechanická karotáž v geologickém průzkumu (Mechanical logging in the geological survey)*, Hodonín: Zemní plyn a nafta, 3, 1973, p, 263 – 272,
292. ZINCHENKO, V, S, – KOZAK, N, M, *Osnovy geofyzičeskich metodob issledovanij*, Moscow: RGGU, 2005,
293. ZINCHENKO, V, S, *Petrofizičeskije osnovy gidrogeologičeskoj i inženernogeologičeskoj interpretacii geofyzičeskich dannych*, Moscow: RGGU, 2005,
294. ZOUBEK, V, *Geologické podklady k projektu údolní přehrady na Vltavě u Zlákovic (Geological bases for a project of a dam reservoir on the Vltava by Zlákovice)*, Praha: NČSAV, 1953, MS,

Material of Organizations and Companies:

1. Code of Practice for Subsurface Exploration for Earth and Rockfill Dams, Bureau of Indian Standards, Doc, WRD 05(451), 2006,
2. Instruktivnyje ukazanija po sostavu i objemu inženernogeologičeskich izyskanij dlja obosnovanija projektov plotin i vodochranilišč, MMIVCH, Kiev, 1975,
3. Inventario de presas espaňolas 1973, Comité Nacional Especial de Grandes Presas, Madrid: MOP, Secretaría general técnica, 1973,
4. Inventario de presas espaňolas 1986, Dirección de Obras Hidráulicas, MOPU, Madrid, 1986,
5. Platné evropské a národní normy (Applicable European and National Standards),
6. Prospekt: Vodní dílo Slezská Harta (Prospectus: Slezská Harta water structure), Povodí Odry (Odra Basin Agency), Ostrava,
7. Prováděcí předpisy k pokynu č, 1 předsedy ČGÚ o IG mapování (Imlementing Regulations to Instruction No, 1 of the President of the Czech Geological Survey on EG Mapping), ČGÚ, Praha, 1970,
8. Směrnice č, 1/1989 o inženýrsko-geologickém mapování (Guideline No, 1/1989 on Engineering-Geological Mapping), ČGÚ, Praha, 1989,
9. Standards for Geological investigations of dam foundations, Japanese National Committee of the International Commission on Large Dams, Tokyo, 1978,
10. The International Journal on Hydropower &Dams, Vol, 15, Issue 1, Sofia, 2008,
11. The International Journal on Hydropower &Dams, Vol, 15, Issue 2, Hanoi, 2008,

Electronic:

1. http://assets,panda,org/downloads/Damsrep,pdf
2. http://cs,wikipedia,org/wiki/Nejvyšší_přehradní_hráze_světa
3. http://cwc,gov,in/Water_Data_Pocket_Book_2006/table10,04Final,pdf
4. http://en,wikipedia,org/wiki/List_of_world's_tallest_dams
5. http://enews,ferghana,ru/article,php?id=2079
6. http://es,wikipedia,org/wiki/Embalse_de_Yesa
7. http://es,wikipedia,org/wiki/Presa_de_Asuán
8. http://es,wikipedia,org/wiki/Presa_de_Tous
9. http://geologie,vsb,cz/inzgeol/sylaby
10. http://ijolite,geology,uiuc,edu/05SprgClass/geo497/class%2013%20/Big%20Dams,htm
11. http://landslides,usgs,gov/learning/majorls,php
12. http://oph,chebro,es/DOCUMENTACION/Congresos/Laderas2007/Vaiont,pdf
13. http://sharif,ir/~ghaemian/DAM_files/File/Course%20Notes,pdf
14. http://subsurfaceevaluations,com/
15. http://voda,arnika,org/Krepsbach

1. http://www,dams,org/
2. http://www,ecgs,lu/pdf/ECGS_CB_Vol16,pdf
3. http://www,embalses,net/pantano-1170-yesa,html
4. http://www,geometrics,com/858-d,html
5. http://www,geomind,eu
6. http://www,infoplease,com/ipa/A0113468,html
7. http://www,itaipu,gov,br
8. http://www,mapygon,com/mexico/michoacan/zitacuaro/presa-el-bosque
9. http://www,miliarium,com/Monografias/TresGargantas/Welcome_def,asp
10. http://www,oyo,com
11. http://www,geotrend,eu
12. http://www,travelatlas,cz/forza/chvojnice/chvojsd,html
13. http://www,water,ca,gov/damsafety/docs/eqc, Standard for Engineering geological Investigation for Dams
14. http://wfraser@water,ca,gov, Fraser W, A, Engineering geology considerations for specifying dam foundation objectives, Division of Safety of Dams, California Department of Safety of Dams, California Department of Water Resources

Index

abrasion	48, 66, 68, 80, 99, 201, 241 – 244, 251, 260, 262, 274, 275, 280 – 283, 287
accumulation	68, 78, 83, 99, 162, 243, 244, 246, 248, 250, 251, 262, 268
arch dam	5, 6, 10, 14 – 16, 21 – 28, 53, 158, 162, 166, 168, 222, 228, 286
cohesion	184, 199, 205, 238, 239, 257
concentrated stress	190, 209, 211, 214, 226, 227
cone penetration test (CPT)	157, 184
construction material	3 – 6, 16 – 18, 26, 27, 29, 30, 40, 44, 46, 48, 59, 69, 73, 76, 79 – 81, 107, 111, 118, 137, 186, 192, 194, 195, 241, 267, 279
core recovery modified (CRM)	188 - 191
corner shear tests	205, 227, 228
curtain	3, 6, 7 - 9, 17, 18, 37, 38, 90, 92, 96, 104, 105, 108, 169 – 171, 211, 237
cut-off wall	9, 88
dam construction	4 – 6, 12, 17, 21, 30, 32 – 34, 36, 37, 43, 47, 53, 56, 57, 65, 114, 166, 211, 266
dam design	3, 24, 25, 37, 129, 198, 263
deformation property	202, 204, 227, 228, 229
documentation of tunnel	172, 173
documented point (DP)	73 – 75, 77, 86
drilling work	9, 63, 114, 152, 156, 255

dynamic (seismic) modulus (E_{dyn})	55, 189, 207, 215, 216, 220, 226, 230, 234, 238, 239
electrical resistivity tomography	116, 124, 125
electron microscopy (SEM)	82, 285
environmental	2, 5, 6, 30, 32, 33, 64, 91, 193, 249
environmental impact assessment EIA	32, 33, 195
erosion	21, 24 – 26, 31, 37, 48, 50, 53, 67, 68, 78 – 80, 83, 85, 93, 99, 102, 110, 159, 161, 173, 194 – 196, 199, 208, 235
	241 – 253, 255 – 259, 261, 262, 268, 272, 273, 275, 277, 280, 282 – 284
exploratory borehole (hole)	61, 86, 99, 112, 153, 154, 164, 168 – 171, 182, 190, 224, 229, 253
exploratory drilling	47, 55, 86, 97
exploratory pit	55, 86, 182, 224
exploratory tunnel	55, 152, 158, 159, 161, 162, 174, 177, 180, 203, 205, 210, 222, 223, 234, 236
exploratory working	43, 49, 55, 63, 120, 151, 156, 159 – 161, 164, 165 – 167, 171, 172, 180, 181, 183, 184, 186, 193, 194, 198, 203, 208,
	222 – 224, 229, 285, 286
flat press	204, 206, 208 – 211
gama logging (GL)	144, 145, 154, 226
gama gama logging (GGL)	114, 140, 141, 143 - 146, 176, 226
geoacoustic measurement	175, 176, 197, 209, 210, 232
geodynamic phenomena	41, 45, 66, 69, 72, 192, 242
geodynamic process	21, 45, 48, 49, 65, 67 – 69, 73, 78 – 80, 192, 242, 269, 275 – 277, 287
geographic information system (GIS)	54, 63, 68, 70, 72 – 74, 77, 91, 107, 285
geomechanical profiling	179, 182
geophysical logging	146, 174, 182
geotechnical model	51, 60, 222
global positioning system (GPS)	70, 73, 283
gravity dam	5, 7, 8, 10 – 14, 16, 24, 25, 27, 28, 36, 53, 98, 101
ground-penetrating radar (GPR)	55, 124, 126, 129
groundwater level (GWL)	66, 79, 90, 92 – 98, 100, 102, 111, 116, 121, 156, 159 - 161, 195 – 197, 212, 275
landslide	18, 21, 22, 29, 31, 48, 68, 69, 72, 76, 80, 83, 110, 111, 114, 171, 172, 236, 241, 243, 244, 246 – 251, 253 – 262, 269
	271, 272, 274, 275 – 284
loading test	158, 202 – 204, 207, 211, 213, 227, 229, 233, 234, 236
modulus of deformation (E_{def})	81, 182, 184, 203, 215, 217, 218, 226, 229, 230, 234, 235
neutron neutron logging (NNL)	143, 144, 185
permeability	2 – 4, 7, 8, 12, 15 – 17, 20, 29, 37, 40, 41, 54, 60, 66, 68, 72, 82, 85 – 92, 96, 102, 105, 108, 110, 123, 140, 160, 164
	168, 171, 192, 201, 202, 211, 212, 229, 232, 233, 235, 241, 252, 259
Poisson’s ratio	176, 204, 207, 208, 216, 217, 224, 234, 238, 239
pressiometric modulus	228, 230

pressiometric tests	153, 191, 227
pumped storage hydroelectric plant (PSHEP)	3, 10, 12, 30, 49, 51, 53, 58, 75, 76, 84 - 86, 93, 100, 122, 133, 145, 163, 173, 185, 202, 209, 225, 230, 235, 236, 250
	251, 252, 273
pumping test	61, 72, 90, 92, 96, 154, 211
resistivity logging (R_{ap})	144, 154, 226
resistivity profiling	19, 81, 108, 112, 115, 116, 120, 121, 123, 127 – 129, 132, 148, 224, 225, 257 – 259
rock quality designation (RQD)	113, 114, 146, 181 – 183, 187 – 191, 219, 220, 226, 240
rose diagram	72, 128, 178, 223
seismic radiography	133 - 135, 147, 190, 210, 217, 220, 224 – 226, 233 – 235
seismic tomography	133, 135, 177
seismicity	28, 31, 39, 45, 59, 73, 74, 194, 195
shallow seismic refraction (SSR)	19, 108, 112, 113, 115, 116, 118, 131, 172
slope failure	18 – 21, 24, 25, 27, 30, 41, 45, 53, 59, 68, 76, 78, 79, 83, 110, 120, 122 – 124, 131, 148, 149, 160, 199, 213, 230, 241
	242, 244, 252 – 255, 260, 263, 272, 275, 277, 279 – 281, 283, 284, 287
slug tests	90, 92, 96, 159, 212
spontaneous polarization (SP)	115, 116, 146, 154
standard penetration test (SPT)	157, 183, 205
stability of banks	4, 242, 276
static modulus	81, 182, 210, 215 – 217, 238, 239
stereoscopic images	54, 248
strength parameters	142, 201, 204, 205, 213, 241, 259,
suffosion	76, 78, 79, 83, 87, 88, 98, 99, 101, 164, 170, 241, 242, 244, 251, 254, 259, 262, 282, 283
swelling	160, 201, 253
TIWAG	203, 211, 227, 229
topographic factors	18, 243
ultrasonic logging	224 – 226
unloading of drill core	208, 210
vertical electrical sounding (VES)	8, 19, 81, 108, 109, 112, 113, 115, 116, 120, 123, 124, 127 - 129, 132, 171, 172, 220, 221
water pressure tests (WTS)	9, 89, 90, 92, 96, 100, 113, 114, 170, 190, 203, 211
water structure	194, 195

List of Places

Place	Country	River	f	l	Page
Akhangaran	Uz	Angren	41.05	70.22	268, 269, 277, 278
Ambuklao	Ph	Agno	16.46	120.74	269
Amistad	Cu	Caonao	21.35	-77.70	87
Andijan	Uz	Karadarya	73.06	40.77	246, 281
Angat	Ph	Angat	14.91	121.16	150, 176, 246, 271, 279 - 284
Arenós	Sp	Mirajes	40.09	-0.55	276
Aswan II	Eg	Nil	32.87	23.97	104, 105, 269, 270
Bakaru	Id		-3.45	119.61	270
Balforsen	Sw	Umealven	65.05	17.23	25
Beleňa	Sp	Sorbe	40.93	-3.19	13, 14
Binga	Ph	Agno	16.39	120.74	269
Boadella	Sp	Muga	42.34	2.83	8, 9
Brno	Cz	-	49.22	16.59	144, 207
Bromme	Fr	Bromme	45.41	2.50	23
Brumovice	Cz	Čižina	50.02	17.76	162, 168
Bubal	Sp	Gallego	42.68	-0.32	276
Budišov	Cz	Budišovka	49.75	17.69	138, 168
Bystrc	Cz	-	49.21	17.21	207
Camarillas	Sp	Mundo	38.34	-1.65	28
Centro Cuba	Cu	Caracusey	21.96	-79.68	12, 26, 51, 52, 58, 75, 75, 84 - 86, 154, 158, 203, 209, 232, 235, 236, 287
Chambas	Cu	Chambas	22.14	-78.94	83
Charco Redondo	Cu	Cautillo	20.26	-76.43	20, 83, 87, 90, 160
Corojo	Cu	Bayamo	20.22	-76.59	20, 67, 68, 83, 87
Correpoco	Sp	Saja	43.17	-4.27	276
Charvak	Uz	Chirchiq	41.63	69.97	83, 110, 242, 281, 282
Dalešice	Cz	Jihlava	49.12	16.12	9, 10, 15, 21, 25, 29, 30, 49, 53 - 55, 82, 158, 162, 163, 167, 168, 173, 174
					179, 183, 185, 186, 189, 198, 220, 222, 225 - 227, 229, 230, 252, 271, 278
Dlouhá Loučka	Cz	Oslava	49.84	17.21	111, 112
Dlouhé Stráně	Cz	Desná	50.08	17.17	133, 134, 145, 176, 252
Dolní Líštná	Cz		49.68	18.69	115, 263
Dolní Lomná	Cz	Lomná	49.55	18.67	162, 168
Dolný vrch	Sl	Turňa	48.58	20.75	148, 149
El Bosque	Me	El Bosque	19.39	-	11, 12, 83

Eroo	Mo	Eroo	49.56	106.90	4
Erdenet	Mo	?	49.08	104.15	262
Fedaia	It	Cordevole	46.46	11.86	25
Foix	Sp	Foix	41.22	1.65	83, 87, 97 - 99, 101
Fortaleza	Cu	Sevilla	-78.35	20.03	87
Fulnek	Cz	Občasný	49.71	17.87	131
Frýdek-Místek	Cz	Morávka	49.60	18.52	81
Genal	Sp	Genal	36.53	-5.26	19, 50, 51, 94, 108, 110, 131, 230 - 232
Goral	Sl	Biela Orava	49.38	19.53	249, 256, 257
Guanayara	Cu	Guanayara	21.96	-80.03	93
Guiamets	Sp	Asmat	41.10	0.75	87, 101, 102
Guisa	Cu	Guisa	20.24	-76.57	21, 82
Hirakud	In	Mahanadi	21.53	83.97	274
Hoštejn	Cz	Březná	49.88	16.76	168
Hracholusky	Cz	Mže	49.79	13.17	246, 254
Hrhov	Sl	Turňa	48.60	20.69	122, 139, 177, 180, 210
Hukvaldy	Cz	Ondřejnice	49.62	18.24	168
Ipel'	Sl	Ipeĺ	48.56	19.72	124, 217, 218
Inguri	Ge	Inguri	42.76	42.03	228, 229
Itaipu	Br+Pa	Parana	-25.41	-54.59	34 - 37, 39
Iskar	Bu	Iskar	42.52	23.54	68, 69, 244, 246
Ivaylovgrad	Bu	Arda	41.63	25.98	68, 69, 244, 246
Jesenice	Cz	Odrava	50.08	12.47	254
Jevišovice	Cz	Jevišovka	48.99	15.98	137
Jigiristan	Uz	Jigiristansay	41.02	70.20	24
Josefův důl	Cz	Kamenice	50.79	15.19	8, 94, 168, 220
Kapchagay	Ka	Ili	43.92	77.10	273
Kariba	Za, Zi	Zambezi	-16.52	28.76	274
Karlov	Cz	Kotelný	50.02	17.26	168
Kirdzali	Bu	Arda	41.63	25.34	68, 69, 244, 246
Krangede	Sw	Ammeran	63.13	16.03	25
Křenovice	Cz	Syrovátka	49.32	17.29	120 - 122
Lubina	Cz	Lubina	49.61	18.19	140, 168, 186
Lučina	Cz	Lučina	49.78	18.42	83, 84, 246
Liptovská Mara	Sl	Váh	49.10	19.49	254
Main Shira	GB	Shira	56.34	-4.97	25

Malá Vieska	Sl	Hornád	48.82	21.23	175
Malpasset	Fr	Reyran	43.51	6.76	22
Melechov	Cz	-	49.61	15.31	143
Mequinenza	Sp	Ebro	41.37	0.27	206
Mingchugur	Uz	Charvat	41.64	70.00	84, 110, 111, 244
Mniší	Cz	Lubina	49.59	18.21	168
Mohelno	Cz	Jihlava	49.10	16.18	10, 30, 31, 147
Morávka	Cz	Morávka	49.58	18.53	29
Moulin Ribou	Fr	Moine	47.02	-0.85	25
Nant-Y-Moch	GB	Nant y Moch	52.46	-3.83	25
Nechranice	Cz	Ohře	50.36	13.42	83, 244 - 246, 252, 254, 281
Nevers	USA	Mississippi	45.41	-92.65	25
Nové Heřmínovy	Cz	Opava	50.04	17.57	31, 168
Nové Mlýny	Cz	Dyje	48.86	16.72	30
Nurek	Ta	Vakhsh	38.37	69.35	97, 274
Nýdek	Cz	Hluchová	49.65	18.75	168
Okhchi	Uz	Sokh	40.31	71.03	263, 264
Olešná	Cz	Olešná	50.13	13.71	246, 254
Orava	Sl	Orava	49.38	19.56	83, 242, 244 - 247, 249, 250, 252, 253 - 256, 258, 260, 262, 268,273
Orlík	Cz	Vltava	49.61	14.18	18
Osada	Sl	Čierná Orava	49.40	19.59	251, 260 - 262
Paso Seco	Cu	Paso Seco	23.05	-81.36	3
Penčice	Cz	Kyjanka	49.52	17.42	168
Pieve di Cadore	It	Piave	46.42	12.39	23, 24
Plachtince	Sl	Plachtinský	48.25	19.28	124
Poduzhemsk	Ru	Kem	64.93	34.53	97
Polhoranka	Sl	Biela Orava	49.42	19.51	249, 251
Pontevedra	Sp	Rons	42.50	-8.62	135, 136
Pozďátky	Cz	-	49.20	15.96	141
Quentar	Sp	Aguas Blancas	37.16	-3.48	15, 16
Rezaksay	Uz	Rezaksay	40.91	71.31	118
Rishtansay	Uz	Rishtansay	40.25	71.28	264
Rogun	Ta	Vakhsh	38.84	69. 92	274
Rozkoš	Cz	Úpa	50.36	16.06	246, 254
Rožnov	Cz	Rožnovská Bečva	49.46	18.18	168
Rožnów	Po	Dunajec	49.76	20.66	242

San Miguel	Cu	Maní Maní	?	?	60
Sanmenxia	Ch	Chuang-che	34.73	111.34	277
Santa Cruz del Norte	Cu	?	23.16	-81.95	188, 190
Sayano-Shishenskaya	Ru	Yenisey	52.83	91.37	272
Selenga	Ru	Selenga	?	?	83
Slezská Harta	Cz	Moravice	49.89	17.58	17, 18, 64, 83, 92, 96 - 98, 128, 147, 160, 162, 168, 183, 216
Slušovice	Cz	Dřevnice	49.27	17.80	119
Susqueda	Sp	Ter	41.98	2.53	16,17
Svarthalsforsen	Sw	Indal	63.12	16.05	25
Šance	Cz	Ostravice	49.51	18.41	171, 246, 254
Štítná nad Vláří	Cz	Zelenský	49.06	17.98	168
Tablachaca	Pe	Mantaro	-12.46	-74.79	83, 284
Teruel	Sp	?	40.36	-1.20	126
Toktogul	Ky	Narynu	41.66	72.64	82
Three Gorges	Ch	Yángzǐ Jiāng	30.82	111.00	31, 274, 276
Topolnitsa	Bu	Topolnitsa	42.42	24.00	68, 69, 244, 246
Tous	Sp	Jucar	39.13	-0.65	10, 11
Trojanovice	Cz	Bystrá	49.52	18.27	168
Tršice	Cz	Olešnice	49.55	17.43	113
Tsonevo	Bu	Kamchia	43.03	27.41	246
Ujala	Cz	Olše	49.88	18.49	114
Usoy	Ta	Marghob	38.27	72.60	275
Ústie nad Priehradou	Sl	Čierná Orava	49.38	19.58	253, 255, 258, 259, 262
Vajont	It	Vajont	46.27	12.33	22, 23, 83, 199, 244, 255, 276, 284
Valmayor	Sp	Aulencia	40.53	-4.05	16, 17
Volta	Gh	Volta	6.30	0.06	274
Vyškov	Cz	-	49.21	16.92	217
Yesa	Sp	Aragón	42.61	-1.18	276, 277
Zbyslavice	Cz	Sezina	49.79	18.07	168
Žamberk	Cz	-	50.07	16.46	132
Žermanice	Cz	Lučina	49.73	18.45	246, 254
Žilina	Sl	Váh	49.21	18.78	134
Almaty - 1	Ka	Aksai	43.11	76.79	266
Almaty - 2	Ka	Bolshaya Almalinka	43.13	76.91	266
Almaty - 3 - Medeo	Ka	Malaya Almalinka	43.15	77.06	266
Almaty - 4 - Minjilki	Ka	Malaya Almalinka	43.08	77.08	266

Almaty - 5- Talgar	Ka	Talgar	43.26	77.21	266
Almaty - 6 - Bolskoe Almatinskoe Ozero	Ka	Bolshaya Almalinka	43.06	76.98	266

Explanation

Br	Brasilia	Fr	France	In	India	Mo	Mongolia	Sp	Spain	Za	Zambia
Bu	Bulgaria	GB	Great Britain	Id	Indosia	Ph	Philippines	Sw	Sweden	Zi	Zimbabwe
Cu	Cuba	Ge	Georgia	It	Italy	Po	Poland	Ta	Tajikistan		
Cz	Czech Republic	Gh	Ghana	Ka	Kazakhstan	Ru	Russia	USA	United States		
Eg	Egypt	Ch	China	Me	Mexico	Sl	Slovakia	Uz	Uzbekistan		

List of figures

Fig.	Name	Page
2.2.1	Geologists and engineers carry out a reconnaissance of a potential dam site on the Eroo	4
2.3.1	Diagram illustrating the transfer of stresses to the bedrock beneath a dam in relation to the hydrostatic pressure exerted on the dam …	6
2.3.2	Different scenarios for the relationship between a dam and the structure of the underlying bedrock	7
2.3.3	Cross section of the site of the dam at Josefův Důl showing the zone of weakened rock beneath the floor of the valley	8
2.3.4	The Boadella Dam an example of a concrete gravity dam	8
2.3.5	Diagram showing the position of the clay wall in the trench excavated in weathered granite relative to the fence of the hole drilled to …	9
2.3.6	View of the Boadella dam under construction	9
2.3.7	Scenario for the construction of an arch dam at Dalešice	10
2.3.8	Schematic geological cross section along the profile of the Mohelno dam with a cross-section showing the structure of the concrete …	10
2.3.9	View of the Mohelno concrete gravity dam	10
2.3.10	Schematic cross section of the valley of the River Jucar. showing the zones of faulting and crushing in the limestone that let to …	11
2.3.11	“Aerial” view of the Tous embankment dam	11
2.3.12	Schematic section across the site of the El Bosque dam showing the position of the buried valleys along the leakage of water took place	12
2.3.13	Profile deemed unsuitable for the construction of the lower dam of the Centro Cuba	12
2.3.14	Diagram illustrating how the forces exerted by the contained water in a multiple slab buttress dam are transferred to the bedrock	13
2.3.15	Cross section showing the structure of the Beleňa multiple slab buttress dam with a conceptual view of the position on the chosen …	13
2.3.16	Diagrams illustrating the principles of construction of a concrete arch dam and the way in which stress is transferred to through …	14
2.3.17	Block diagram showing the zones of weakness governing the geotechnical conditions in the proposed excavation for the construction …	15
2.3.18	View showing the combined concrete and rock fill gravity dam at Dalešice under construction	15
2.3.19	Cross section showing the central profile and foundation of the Quentar concrete arch dam with a view of the dam under construction …	16
2.3.20	View of the Susqueda arch dam under construction in 1966. The Susqueda dam is a three centred. double curvature arch with a height …	16
2.3.21	Schematic cross section of the Susqueda arch dam together with a photograph showing the steep support of the foot of the arch	17
2,3,22	Oblique satellite view looking west to the Valmayor rockfill embankment dam	17

2.3.23	Cross section showing the internal structure of the Slezska Harta rockfill embankment dam	18
2.3.24	Cross section along the profile originally chosen for the Orlík dam showing the rotational slope failure above the Vltava and the …	18
2.3.25	Section along the Genal P3 profile showing the interpretation of the underlying geology based on the results of seismic …	19
2.3.26	Geological section along the profile chosen for the Corojo III dam. showing the underlying landslide	20
2.3.27	Cross section showing the extensional fractures and unstable blocks of limestone on the profile of the Chargo Redondo dam	20
2.3.28	Landslide of Paleogene limestone on Cretaceous tuffites at the Guisa dam site	21
2.3.29	View of a landslide at Dalešice	21
2.3.30	Malpasset dam. after 50 years	22
2.3.31	View of the Vajont Dam. showing the scar produced on Monte Toc above the reservoir as a result of the catastrophic landslide …	23
2.3.32	Cross section showing the geological configuration of the landslide behind the Vajont dam	23
2.3.33	Profile along the Bromme dam	23
2.3.34	Profile along the Pieve de Cadore dam	23
2.3.35	Landslide threatening dam	24
2.3.36	Common topographic and geological scenarios for dam sites	26
2.3.37	Geological section along the profile for the downstream reservoir of the Centro Cuba PSHEP on the River Caracusey	26
2.3.38	Geological cross section parallel to drainage showing the setting of the Camarillas dam in the relation to the uplift caused by …	28
2.3.39	View of the impermeable asphaltic concrete cover on the upstream face of the Morávka rock-fill embankment dam before …	29
2.3.40	View of the landslide that took place on the margin of a stone quarry in the backwater area of the Dalešice dam	29
2.3.41	Diagrammatic cross sections showing the domain of influence of dam construction work on the surroundings geological environment	30
2.3.42	Schematic plan depicting the position of the Mohelno and Dalešice dams in relation the surroundings topography. settlements …	31
2.4.1	Itaipu dam	34
2.4.2	Construction of Itaipu dam	35
2.4.3	Geological map of Itaipu lake surroundings	36
2.4.4	Detailed geological map of Itaipu dam	36
2.4.5	Cross section of Itaipu main dam	37
2.4.6	Breccia of the footing beneath	38
2.4.7	Side repair of the bad mechanical properties	38
2.4.8	Shape of the bottom of the Itaipu lake	39
3.3.1	Extract from the 1:200 000 scale map sheet showing the geology in the area of the Dalešice PSHEP	49
3.3.2	Interpretation of the pattern of drainage in the Genal river basin in terms of the main structural lineaments	50
3.3.3	Map showing the drainage and topography of the area surrounding the reservoirs of the Centro Cuba PSHEP	52
3.3.4	Example of a detailed geological map. originally at a scale of 1:1 000. showing the rock and the structures at the Dalešice da site	53
3.3.5	Perspex model of the Dalešice dam site	54
3.3.6	TV image of a cavity with planes of weakness and their azimuths projected in yellow	55
3.3.7	View of the cutting made along the foot of the right bank at Dalešice showing details of the geological profile and the measurements …	55
3.5.1	Geological map and section of the San Miguel dam profile showing the location of planned boreholes	60

3.5.2	Geological cross section along the right bank of the River Moravice at the position of the Slezská Harta dam showing the ration ship …	64
4.1.1	Example of the delineation of an engineering-geological region I. sub-regions A and B and areas A1. A2. B1 and B2 in the vicinity …	68
4.1.2	Maps showing the engineering-geological zones on the slopes surrounding four water reservoirs in Bulgaria	69
4.2.1	An example of an engineering-geological map	71
4.3.1	View of an abandoned well with the log describing the sedimentary sequence and physical parameters	78
4.3.2	Cross section and geophysical profile showing how measurements of resistivity were used to interpret the engineering geology …	81
4.4.1	Geological cross section along the profile of the Nýdek Dam illustrating the complexity of the geological structure	83
4.4.2	Schematic geological map and cross section along the drainage of the Foix dam	83
4.4.3	Geological cross section along the profile of the Lučina dam showing the unfavourable conditions of the slope stability and permeability	84
4.4.4	View of a slope failure at Mingcukur on the banks of the Charvak dam in tension cracks	84
4.4.5	General view across the area of the Centro Cuba PSHEP showing the location of the planned penstock and power house	84
4.4.6	Geological map and cross section showing the rocks and structures in the area of the Centro Cuba PSHEP	86
5.1.1	Geological sketch map of the area of the Slezka Harta dam	92
5.1.2	Geological cross section A-A' in the area between the River Moravice and the Lesná brook with the projected profile with the bed …	93
5.1.3	Sketch map showing the network of drainage in the upper basin of the Guanayara River in relation to the tectonic lineaments in the area	93
5.1.4	Plan showing the flow pattern of groundwater determined by geoelectrical measurements in the vicinity of a test pit at the Josefův Důl …	94
5.1.5	TV image captured showing stones thrown into the bottom	94
5.1.6	TV images captured during inspection of borehole casing	95
5.2.1	Map of the hydroisohypses on the piezometric surface on the right slope of the River Moravice showing the pattern of drillholes used …	97
5.2.2	Graph showing the relationship between the level of water in the reservoir behind the Fox dam and the rate of the loss of water from …	99
5.2.3	Geological cross section showing the scaleof development of karst cavities in the limestone above the River Caracusey	100
5.2.4	View of the karst topography in the limestone at Caracusey showing a cavity large enough for a man to enter	100
5.2.5	View of the Guiamets dam in the Ebro Basin. Sp. showing grouting in progress on the left flank of the dam behind the abutment	102
5.2.6	Plan and geological sections showing the layout of the Guiamets dam	102
5.2.7	Logs of magnetic susceptibility demonstrating the progressive improvement in cementation of permeable horizons as a result of two …	103
5.2.8	Geological section across the Nile valley at the site of the Aswan dam showing the grout curtain injected into the sequence …	104
5.2.9	View of the Aswan High Dam	104
6.1.1	Genal dam profile. a survey of the left slope	108
6.1.2	Cross-section of the Mingchukur landslide on the bank of the Charvak reservoir	111
6.1.3	Survey for a quarry at the Dlouhá Loučka dam	112
6.1.4	Survey for the hydraulic facilities at the Tršice dam site	113
6.1.5	Geophysical logs used to check a borehole at Ujala	114
6.1.6	Dolní Líštná - detection of seepage through a dam body	115
6.2.1	Seismic survey refraction along profiles	118
6.2.2	Seismic refraction survey of a profile along/across the earth-fill dam on the River Rezaksay showing the effect of compression …	118
6.3.1	Early geophysical survey of the Slušovice dam site	119

6.3.2	Geophysical survey of a valley near Křenovice	121
6.3.3	Survey of the area for sitting the Hrhov dam and power plant	122
6.3.4	Landslides on the slope of the Plachtince dam profile	124
6.3.5	Method of eletrical resistivity tomography	125
6.3.6	“OhmMapper” method	126
6.3.7	Ground-penetrating radar	126
6.3.8	Radar detection of a cavity	127
6.3.9	Azimuthal VES	128
6.3.10	Anisotropy based on resistivity profiling	128
6.3.11	Fracturing of the rock mass indicated in the field of velocities measured by SSR	131
6.3.12	Silicification of claystones at Žamberk	132
6.3.13	Change of velocity with depth	133
6.3.14	Seismic radiography - fan graphs	133
6.3.15	Tomographic processing of seismic radiography. Dlouhé Stráně PSHEP	134
6.3.16	Seismic tomography between boreholes at Žilina	135
6.3.17	Tomographic check of the dam body at Pontevedra	136
6.3.18	Ultrasonic measurement on drill core	137
6.3.19	Ultrasonic log of a boreholes	138
6.3.20	Ultrasonic check of concrete buttresses	139
6.3.21	Ultrasonic measurements made on a test block of limestone	139
6.3.22	Geophysical logs of borehole J51 at Větřkovice	140
6.3.23	Accoustig borehole-wall imaging	141
6.3.24	Optical borehole-wall imaging	143
6.3.25	Comprehensive logging using nuclear methods for measurement in drill rods	144
6.3.26	Logging measurements in perforated casings	145
6.3.27	GGL in shallow boreholes	145
6.3.28	Flow of water across a borehole	145
6.3.29	Magnetometric survey at the Mohelno dam	147
6.3.30	Measurement of magnetometric gradient	148
6.3.31	Gravimetric profile across a steep slope in limestone	149
6.3.32	Gravimetric measurement across a zone of weakness	150
6.3.33	Temperature measurements in the Macua - Umiray tunnel	151
6.3.34	Measurement of alpha tracks	151
7.1.1	Error in lithological description of core detected by logging	154
7.1.2	Damage to borehole casing	155
7.1.3	Comprehensive graphic borehole log	155

7.2.1	Mouth of tunnel G183	158
7.3.1	Relation between borehole location and GWL	160
7.3.2	Relation between location of borehole and impermeable basement	160
7.3.3	Pattern of boreholes across steep geological contacts	161
7.3.4	Layout of boreholes and tunnels in a mountain valley	161
7.3.5	Layout of exploratory workings at the Dalešice dam	162
7.3.6	Survey work in the Dalešice dam profile for the option of an arch dam	163
7.3.7	Delineation of the area for a preliminary survey	165
7.3.8	Delineation of the area for a detailed survey	166
7.3.9	Relationship between dam height and the number of boreholes and pits	167
7.3.10	Relationship between dam height and the number of boreholes and pits	167
7.3.11	Relationship between dam height + maximum and average depths of boreholes	168
7.3.12	Maximum distances between boreholes	169
7.3.13	Pattern of lines of same pressures	170
7.4.1	Parametric measurements in a dug pit	172
7.4.2	Geophysical logging of a tunnel. showing the method used in the late 1960s	174
7.4.3	Comprehensive geophysical logging of a tunnel	175
7.4.4	Log of bulk density along a tunnel	176
7.4.5	Measurement of temperature and properties of water inflows along a tunnel	176
7.4.6	Comprehensive logs of a pit	177
7.4.7	Photographic documentation of a tunnel wall	179
7.4.8	Complete description of a tunnel	180
7.4.9	Geophysical measurements in and above a tunnel	182
7.4.10	Comprehensive logs of a borehole	183
7.4.11	Susceptibility of a drill core	183
7.4.12	Dynamic penetration test	184
7.4.13	Static penetration combined with logging	185
7.4.14	Diversion tunnels and channel at Dalešice	186
7.4.15	Log of a diversion tunnel	186
7.4.16	Observation and description of the base of a dam footing	187
7.5.1	Relationship between RQD and apparent resistivity and velocity of longitudinal waves	189
7.5.2	RQD. WPT. and V	190
7.5.3	CRM in a geological cross section and on a map of a construction site for a power plant	191
8.3.1	Laboratory tests on rock samples	202
8.3.2	Loading bridge being used to make a deformation test in the field	203
8.3.3	Geotechnical tests in a tunnel. right – detail of sensors	203

8.3.4	Preparation of a test with flat presses	204
8.3.5	Shear test in a frame: state before and after the test	204
8.3.6	Large-area shear test	206
8.3.7	Results of geophysical measurement in a mechanically driven working	207
8.3.8	Measurement of displacements in flat presses	208
8.3.9	Seismic identification of an arch	209
8.3.10	Geoacoustic changes and changes in moduli of elasticity	210
8.4.1	Relationship between static and dynamic moduli	215
8.4.2	Relationship between V and E at Slezská Harta	215
8.4.3	Poisson's ratio – seismic measurement	216
8.4.4	Determination of Poisson's ratio in a borehole	216
8.4.5	Distribution of the coefficient of fracturing and the moduli of deformation at the Ipel' site	218
8.4.6	Relationship between the velocity of longitudinal waves and strength	218
8.4.7	Relationship between porosity and the modulus of elasticity and strength	219
8.4.8	Relationship between RQD and resistivity and the velocity of longitudinal waves	219
8.4.9	Distribution of velocities and apparent resistivity. and the coefficient of fracturing	220
8.4.10	Relationship between ρ_a and E_{dyn}	220
8.4.11	Distribution of dynamic moduli of elasticity in the Josefův Důl dam profile	220
8.5.1	Longitudinal section of the intake. penstock and pumped storage hydroelectric plant ...	225
8.5.2	Velocity maxima below the Dalešice valley bottom	226
8.5.3	Velocity contours thirty metres below the Dalešice valley	227
8.5.4	Large-area two-directional pressure test	227
8.5.5	Corner shear test	228
8.5.6	Aerial view of the Dalešice dam	230
8.5.7	Variations in resistivity and velocities of longitudinal waves	233
8.4.8	Concentration of stress. and geoacoustic activity	233
8.5.9	Growth of velocities and moduli	234
8.5.10	Relationships of modulis	234
8.5.11	Relocation of the cavern of a powerhouse	236
9.1.1	Co-action of erosion and abrasion	243
9.1.2	Erosion of accumulations below abrasion cliffs	244
9.1.3	Landslides on the banks of the Orava reservoir	245
9.1.4	Degradation of banks on the Nechranice reservoir	245
9.1.5	Strengthening of banks with stone packs	247
9.1.6	Plan of the Orava reservoir	247
9.1.7	Recession of the river bank line	247

9.1.8	Pattern of the recession of the base of a bank cliff	248
9.1.9	Devastation of a marina	248
9.1.10	New protection of banks by gabions	248
9.1.11	Frequencies of levels on the Orava and Angat reservoirs	249
9.1.12	Beach abrasion platforms	250
9.1.13	Prediction of the development of bank recession	250
9.1.14	Graph of bank recession	251
9.1.15	Prediction of bank recession	251
9.2.1	The Vajont Dam. left – a satellite view	255
9.2.2	Map and cross section of Goral landslide	256
9.2.3	Goral landslide	257
9.2.4	Ústie nad Priehradou landslide	258
9.2.5	Suffosion sinkhole	259
9.2.6	Crumbling and collapse of abrasion walls	260
9.2.7	Development of sliding at Osada	260
9.2.8	Osada site. section and photo	261
9.2.9	Erosion furrows	261
9.2.10	Abrasion cliffs	262
9.2.11	Gouging of bank by sliding ice block	262
9.3.1	Dam of Erdenet deposit	262
9.3.2	Protective dam on the River Sokh	263
9.3.3	Protective dam on the River Burbaliksay	263
9.3.4	Protective dam for small rivers	264
9.3.5	Protective barrier against erosion	265
9.3.6	Protection of the City of Almaty from debris flow	266
9.4.1	Sedimentation and erosion on the Orava reservoir	268
9.4.2	Sedimentation on the Akhangaran reservoir	269
9.4.3	Dejection cone on the Akhangaran reservoir	269
9.4.4	Functional dam and non-functional powerhouse	270
9.4.5	Time course of sedimentation in the Bakaru reservoir	270
9.4.6	Release of reservoir sediments	271
9.4.7	Reservoir bottom view by 3D sonar	271
9.4.8	Accident at Sayano-Sushenskaya dam. 17 August 2009	272
9.4.9	Time course “Kapchagay delta” formation	273
9.5.1	Usoy natural dam and landslides on its banks	275
9.5.2	Landslides on the Yesa dam	277

9.5.3	Landslide “Verkhneturkskiy” on the bank of the Akhangaran dam	278
9.5.4	Movement of the Verkhneturkskiy landslide	278
9.5.5	Silted tunnel at the Angat hydraulic facility	279
9.5.6	Renewed growth of the vegetation cover after the typhoon Nanmadol	279
9.5.7	Vegetation cover on slope deformations	280
9.5.8	Landslides below the line of the maximum water level	280
9.5.9	Typical morphology of “terracettes” on reservoir banks	281
9.5.10	Settlement on landslide next to reservoir	282
9.5.11	Suffosion on the banks of the Charvak (left) and Angat (top and right) reservoirs	282
9.5.12	Landslide on the Angat reservoir and sonar measurement	283

List of tables

Table	Name	Page
3.3.1	Categories of areas based on the complexity of engineering-geological conditions	45
3.3.2	Categories of complexity of structure based on technical characteristics	46
3.4.1	The stages of the engineering design and EG survey of dam projects	57
3.4.2	Division of survey into stages	58
4.1.1	Categories of information depicted on EG maps and the corresponding survey and design *stages*	66
4.2.1	Number of DP on an EG map	74
6.2.1	Choice of geophysical methods	116
7.3.1	Borehole spacing and depth in relationship to designed dam height	164
7.3.2	Recommended average spacing of exploratory holes in surveys for dams	164
7.3.3	Recommended average depths of exploratory workings in EG surveys for dams	165
7.3.4	Coefficients	166
7.3.5	Maximum recommended depths of boreholes used in EG surveys for dams	168
7.5.1	Evaluation of rocks using values of RQD and CRM	188
7.5.2	Relationship between RQD and ρa in granulite at Dalešice	189
8.1.1	The optimal sequence of stages of Investigation	194
8.3.1	Types of geotechnical test	210
8.5.1	Physical properties of rocks as a basis for selection of surface methods	223
8.5.2	Comparison of survey work at dams	229
8.5.3	Comparison of moduli	230
8.5.4	Measurements of moduli of elasticity made on drill cores and in situ	231
8.5.5	Concise summary of essential information about dam profiles on the River Genal	232

8.5.6	Relationship between modulus of deformation and dynamic modulus of elasticity	234
8.5.7	Recommended values of moduli	235
8.5.8	Comparison between moduli of elasticity assessed by WPT and by seismic radiography	235
8.5.9	Physical and mechanical properties- I	238
8.5.10	Physical and mechanical properties- I1	239
8.5.11	Geotechnical properties	240
9.1.1	Intensity of development of exogenous processes in loamy sediments	244
9.1.2	Characteristics of selected dam reservoirs and their banks	246
9.2.1	Types of slope deformations at dam reservoirs in the Czech and Slovak Republics	254

Content

1	Introduction			1
2	Basic Criteria of Dam Design			3
	2.1	Function of Dams		3
	2.2	Selection of a Site for Dam Construction		4
	2.3	Determining Factors for the Selection of a Site and Type of Dams		5
		2.3.1	Geological Factors of the Selection of a Dam Site	6
		2.3.2	Topographic Factors in the Selecting a Dam Site	18
		2.3.3	Effect of Climatic Conditions on the Selection of Types of Dams	27
		2.3.4	Seismic Effects	27
		2.3.5	Construction Materials	29
		2.3.6	Ecological Factors Affecting the Construction of Reservoirs	30
		2.3.7	Environmental Impact Assessment (EIA) in Relation to Construction and Operation …	32
	2.4	The Itaipu Dam as an Example of How to Solve Problems of Large Dams		34
3	Methods Used to Carry Out an Engineering-Geological Survey			40
	3.1	Tasks of the Engineering-Geological Survey		40
	3.2	Rules Governing the Procedures Used in an Engineering-Geological Survey		41
		3.2.1	Rule of Step-by-Step Procedures in Carrying Out an Engineering-Geological Survey	41
		3.2.2	The Rule of Comprehensiveness of Engineering-Geological Survey	42
		3.2.3	Accuracy. Detail and Economy of an Engineering-Geological Survey	43
	3.3	Procedure of Carrying Out an Engineering-Geological Survey		43
		3.3.1	Choice of a Survey Method	44
		3.3.2	Significance of Engineering-Geological Survey in Preparatory Work and Construction	46

		3.3.3	Basic Strategy for an Engineering-Geological Survey	48
	3.4	The Planned Stages of an Engineering-Geological Survey		56
	3.5	Technical Specifications and Project of Survey Work		58
		3.5.1	Technical Specifications	58
		3.5.2	Designing the Geological Survey Work	60
4	Engineering-Geological Mapping			65
	4.1	Definition of Tasks of Engineering-Geological Mapping		65
	4.2	The Compilation and Presentation of an Engineering-Geological Map		69
	4.3	Work Flow for Compilation of Engineering-Geological Maps		74
		4.3.1	Preparation Work	75
		4.3.2	Fieldwork	77
		4.3.3	Laboratory Work	81
	4.4	Engineering-Geological Mapping of Special Areas		82
5	Hydrogeological Survey			87
	5.1	Preliminary Hydrogeological Survey		88
	5.2	Detailed Hydrogeological Survey		95
		5.2.1	Hydrogeological Survey for Preliminary Design	96
		5.2.2	Hydrogeological Survey for Detailed Design	101
6	Geophysical Surveys			106
	6.1	Tasks of a Geophysical Survey		107
	6.2	Choice of Geophysical Methods		116
	6.3	Use of Geophysical Methods		119
		6.3.1	Geoelectrical Methods	123
		6.3.2	Seismic Methods	129
		6.3.3	Logging Methods	140
		6.3.4	Other Geophysical Methods	147
7	Direct Survey Work			152
	7.1.	Drilling Surveys		152
	7.2	Mining Work		157
	7.3	Scope of Subsurface Exploration Work		159
	7.4	Comprehensive Documentation of Exploratory Workings		171
		7.4.1	Documentation of Tunnels and Pits	172
		7.4.2	Description of Exploratory Boreholes	182
		7.4.3	Observations Made during Construction	184
	7.5	Application of RQD for Geotechnical Evaluation of the Rock Mass		187
	7.6	Trends of Direct Survey Work		191

8	Geotechnical Surveys		192
	8.1	General Rules for Geotechnical Surveys	192
	8.2	The Rock Environment as a Determining Factor in Dam Design	198
	8.3	Methods Used for Geotechnical Surveys	201
		8.3.1 Identification of Deformation Properties	202
		8.3.2 Detection of Strength Parameters	204
		8.3.3 Measurement of the State of Stress in the Rock Mass	208
		8.3.4 Tests of Permeability	211
		8.3.5 Final Report on the Results of a Geotechnical Survey	212
	8.4	Correlations between Parameters	213
		8.4.1 Causal Relationships	213
		8.4.2 Statistical Relationships	219
	8.5	Practical Examples	221
9	Engineering-Geological Survey of the Reservoir Areas		241
	9.1	Modification of Reservoir Banks	242
	9.2	Modification of Banks and Slope Failures	252
		9.2.1 The Area of Landslides at Goral	256
		9.2.2 The Landslides at Ústie nad Priehradou	258
		9.2.3 The Landslides at Osada Site	260
	9.3	Dams for Special Use	262
	9.4	Silting of Water Reservoirs	268
	9.5	Examples of Phenomena Affecting the Banks of Water Reservoirs	274
10	Conclusion		285
	Abbreviations		289
	References		290
	Index		304
	List of places		307
	List of figures		311
	List of tables		318
	Contents		319

"A BAD-BEHAVED DAM AND RESERVOIR"
The killer (caused by a landslide), 3,500 casualties
Vajont, Italy