The Canadian Geotechnical Society
La Société canadienne de Géotechnique

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Introduction of Geotechnical Program

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It is a great honour and a privilege to open the Geotechnical program of the Canadian Engineering Centennial Convention. This program covers the history of our profession, geotechnical practice in construction, mining, dam building, transportation, waste management, energy development, and looks at the future of the profession. These are reported in thirteen papers of the proceedings.

Soil mechanics or geotechnique has played a very important role in the development of Canada. For example, before it was recognized as a major engineering discipline, it was applied in the construction of the Canadian Pacific Railway and the construction of the Rideau Canal. Since then, it was practiced in the design and construction of the Saint Lawrence Seaway, hydro power developments, the Trans Canada Highway, national port facilities, Canadian airports, tunnels, and the foundations of many engineering structures.

The first formal soil mechanics papers in Canada were presented by G.B. Williams and R. Peterson at a national conference organized by the Engineering Institute of Canada in 1945 (Eden and McRostie, 1987). (Mr. Peterson has since passed away but Mr. Williams was present at the Centennial Convention). The Associate Committee on Geotechnical Research of the National Research Council was also formed in 1945 with R.F. Legget as Chairman.

It organized the first Canadian Soil Mechanics Conference in 1947, and continued to do so until 1961 when it was assisted by the Geotechnical Engineering Division of the Engineering Institute of Canada. In 1963 the Geotechnical Engineering Division took full responsibility for the conferences until 1972 when they were taken over by the newly formed Canadian Geotechnical Society. Since 1945, annual geotechnical conferences were held every year except for 1965 when Canada hosted the Sixth International Conference on Soil Mechanics and Foundation Engineering in Montreal. Annual geotechnical conferences have been held in Vancouver, Banff, Lethbridge, Edmonton, Calgary, Saskatoon, Regina, Winnipeg,

J'ai le grand honneur et le privilège de présenter le programme de géotechnique du Congrès du centenaire du génie canadien. Ce programme traite de l'histoire de notre profession et de l'application de la géotechnique dans les domaines de la construction, de l'exploitation des mines, de l'érection des barrages, du transport, de la gestion des déchets et de l'exploitation énergétique, et il donne un aperçu de l'avenir de cette profession. Le compte rendu comporte treize communications portant sur ces questions.

La mécanique des sols, ou géotechnique, a grandement contribué au développement du Canada. Par exemple, avant que la géotechnique ne soit considérée comme une branche du génie, on l'a appliquée lors de la construction du chemin de fer du Canadien Pacifique et du canal Rideau. Depuis lors, on a eu recours à la mécanique des sols pour concevoir et construire la voie maritime du Saint-Laurent, les aménagements hydro-électriques, l'autoroute transcanadienne, les installations des ports nationaux, les aéroports canadiens, les tunnels et les fondations d'une multitude d'ouvrages de génie civil.

Les premières communications officielles sur la mécanique des sols à être présentées au Canada l'ont été par G.B. Williams et R. Peterson lors d'une conférence nationale organisée par l'Institut canadien des ingénieurs en 1945 (Eden et McRostie, 1987). (M. Peterson est décédé depuis mais M. Williams était présent au Congrès du centenaire). Le Comité associé de recherches géotechniques du Conseil national de recherches a lui aussi été formé en 1945, sous la présidence de R.F. Legget.

Le comité a organisé en 1947 la première conférence sur la mécanique des sols et il a exercé cette fonction jusqu'à ce qu'il reçoive, en 1961, l'aide de la Division de géotechnique de l'Institut canadien des ingénieurs. En 1963, la Division de géotechnique a assumé toute la responsabilité des conférences jusqu'en 1972, année à partir de laquelle la Société canadienne de géotechnique s'en est occupée. À partir de 1945, on a tenu des conférences de géotechnique

Niagara Falls, Toronto, Kingston, Ottawa, Montreal, Quebec City, Fredericton, and Halifax. The 40th Annual Conference will be held in October, 1987 in Regina.

Since we started 40 years ago, we have grown to become the third largest Geotechnical Society in the world, among a family of 57 national societies making up the international Society on Soil Mechanics and Foundation Engineering. The membership of the Canadian Geotechnical Society is about 1200, which represents about one half of all geotechnical engineers in this country. Our Canadian Geotechnical Journal which started in 1963, is one of the foremost geotechnical journals in the world. The most important factor that will affect the future of our profession however, lies in the research we must do and in the high standard of education we must maintain.

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tous les ans sauf en 1965, alors que le Canada a été l'hôte du sixième Congrès international de mécanique des sols et des travaux de fondations, qui a eu lieu à Montréal. Depuis, on a tenu des conférences de géotechnique à Vancouver, Banff, Lethbridge, Edmonton, Calgary, Saskatoon, Regina, Winnipeg, Niagara Falls, Toronto, Kingston, Ottawa, Montréal, Québec, Fredericton et Halifax. Le 40^{e} congrès annuel aura lieu à Regina en octobre 1987.

En 40 ans d'existence, la Société canadienne de géotechnique, l'une des associations nationales formant la Société internationale de la mécanique des sols et des travaux de fondations est devenue la troisième plus grande au monde. Elle compte quelque 1 200 membres, c.-à-d. environ la moitié des ingénieurs géotechniciens du pays. La Revue canadienne de géotechnique, qui a fait ses débuts en 1963, est l'une des revues de géotechnique les plus prestigieuses du monde. Cependant, l'avenir de notre profession repose sur la qualité des programmes de recherche et d'éducation que nous mettrons en oeuvre.

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A Profile of the Canadian Geotechnical Profession and its Practice

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SUMMARY

The geotechnical profession in Canada is characterized.

There are 2400 geotechnical practitioners in Canada. 50% work as consultants. Work is related to natural resource and energy development, industrial development, and construction associated with population centres.

Differences between geotechnical engineering and other types of engineering are identified. Most engineers work in a man-made environment or one that they can specify. The geotechnical engineer works with the earth. He must explore to determine what is there, test to determine material properties, and assess the interaction between site conditions and operations. Follow-up is an essential part of the predictive process.

Most geotechnical practitioners have received advanced education. Statistically, the breakdown of university degrees is 99% bachelor's, 80% master's and 21% PhDs.

Geotechnical engineers have a special need to share case histories and for further education. The Canadian Geotechnical Society has developed to meet many of these needs. The profession and its society are quite dependent on each other.

RÉSUMÉ

L'article donne les caractéristiques de la profession géotechnique au Canada.

Près de 2400 géotechniciens pratiquent au Canada, dont 50% sont ingénieurs-conseils. Leur travail est relié à l'exploitation des richesses naturelles et énergétiques, aux développements industriels et aux travaux de construction dans les centres urbains.

Les différences entre le génie géotechnique et les autres types de génie sont identifiées. La plupart des ingénieurs travaillent dans un environnement créé ou contrôlé par l'homme. L'ingénieur géotechnicien travaille avec la terre. Il doit l'explorer pour la caractériser, la tester pour en déterminer les propriétés, et évaluer les interactions entre les conditions du site et les travaux. Le suivi est une partie essentielle du processus de prévision.

La plupart des géotechniciens ont une formation avancée. Statistiquement, on compte 99% de diplômés en génie, 80% ayant une maîtrise et 21% un doctorat.

Les ingénieurs géotechniciens éprouvent un besoin particulier d'échanger de l'information sur des études de cas ainsi qu'un besoin de formation continue. La Société canadienne de Géotechnique s'est développée pour satisfaire plusieurs de ces besoins. La profession et sa Société sont étroitement liées.

INTRODUCTION

This paper describes the Geotechnical Profession in Canada and its practice.

Canada, with its large area, offers many types of terrain and challenges to development. The geotechnical profession plays an important role in guiding orderly development in Canada. They work with the ground and link the works of man to the earth. They explore and interpret ground conditions relevant to:

- site selection
- site development and best use
- the most appropriate method of site development, mining or construction
- monitoring and remedial works
- reclamation and restoration

Often, the geotechnical professional works with a team of other practitioners - designers, builders, miners and environmentalists.

A PROFILE OF THE CANADIAN GEOTECHNICAL PROFESSION

Numbers and Background

There are approximately 2400 Geotechnical Professionals in Canada. The breakdown of membership in the Canadian Geotechnical Society, which is the main society representing the profession, is as follows:

- Total Canadian Membership:	1210
- Engineering Geology Division:	290
- Rock Mechanics Division:	187

The level of education of full members of the Canadian Geotechnical Society is shown below:

- Bachelor's Degree:	99%
- Master's Degree:	80%
- PhD Degree	21%

Distribution

The distribution of membership in the Canadian Geotechnical Society is shown below:

- British Columbia:	131
- Alberta:	240
- Saskatchewan:	110
- Manitoba:	67
- Ontario:	388
- Quebec:	220
- Atlantic Provinces:	54
- International:	91
- TOTAL:	1301

Geotechnical practitioners tend to live where the action is or where client design services are located. Opportunity reflects industrial or natural resource development and construction activity associated with population centres. Many who reside in the major population centres service remote areas and international clients.

The highest concentration of geotechnical engineers per capita exists in Alberta. The concentration reflects the extent of geotechnical involvement in energy-related projects in Alberta and in frontier areas.

Employment

The following summarizes how Canadian Geotechnical Engineers in Canada are employed:

- Consulting:	50%
- Government:	25%
- Education:	15%
- Industry:	10%
- TOTAL:	100%

Half of the profession practice through independent consulting firms. Many of the practitioners in industry and government offer services as if they were in-house consultants.

Source of New Technology

Some aspects of geotechnical engineering are imported, but much of it is regional and site specific.

Geotechnical principles have been utilized in Canada from the earliest days. It has only been actively applied as a separate discipline since the late 1950s. In the past two decades, there has been extensive development of new geotechnical technology pertinent to Canada - especially for Northern, frontier and energy developments. The new technology is usually developed for specific applications or projects. Because of this development, Canada's practitioners are world leaders in their own right.

GEOTECHNICAL PRACTICE - HOW IT DIFFERS FROM OTHER ENGINEERING

Consulting Companies

Many engineering disciplines band together and form large companies that offer integrated services.

In contrast, Geotechnical Engineers tend to stick together and form consulting firms that specialize in Geotechnical Engineering. They join forces to share expertise, to share special facilities, and to serve many clients. The special facilities include:

- Computers
- Geotechnical Library
- Geotechnical Laboratory
- Drilling/Exploration Equipment

Much of the work undertaken by the large geotechnical consulting firms is on international assignments.

Some highly experienced Geotechnical Engineers practice alone. Much of their work is of an advisory nature.

A minimum of several professionals is needed to sustain a viable consulting practice. The average numbers of core professionals is 40.

Work Environment

Most engineering disciplines work with a man-made or controlled environment.

In contrast, the Geotechnical Engineer works with natural conditions at the site chosen for development. That means he has to explore to find out what is there. Then he has to test to determine strength characteristics of the natural materials. Finally, he has to interpret the interaction of those properties with the proposed site development.

Tools have been developed for exploring subsurface conditions. They are tested to the extreme when the site lies in a disturbed urban area, in a steamy jungle setting, or at the bottom of the ocean.

Working Materials

Most engineering disciplines are able to work with manufactured materials that have definable properties. Part of the design includes specifying the material properties.

The geotechnical engineer works with the earth. After he determines what the native materials are, he has to perform tests to determine relevant material properties. Then he has to predict how the materials will affect site development and vice versa. A wide range is possible in:

- types of material present
- properties of those materials
- behaviour under site development and working conditions
- potential for unexpected conditions or departures from design assumptions.

Need For Follow-up

Due to uncertainties inherent in the exploration and appraisal process, it is important that the geotechnical engineer have an opportunity to follow up during construction and operations. The basic purpose of a geotechnical inspection role during construction or operations is to:

- Verify that ground conditions exposed during construction or operations are the same as were assumed in design
- If conditions are different, assess the impact and take corrective action
- Monitor construction/operating practice to see if it conforms with design intent
- Assess the interaction between operations and the natural ground conditions to see if it conforms with design intent
- Monitor ongoing performance.

The process of correcting design or operations by adjusting to observed field behaviour is known as the observational approach.

Role

The role of the geotechnical engineer in a given project varies with the relative importance of his input. The full role that would be expected in a large scale natural resource project is summarized below:

- A clear understanding of the final project desired, the development options, and the design/construction/operating options
- Scope to explore pertinent site conditions adequately
- Interact with the rest of the project team to understand the consequence of each, design/development alternative, and the risk involved
- Work with the project team to identify the optimum approach to the project, taking project risk into account
- Provide effective input into project development and design guidelines
- Provide effective input into construction plans and specifications
- Have an effective monitoring role during construction
- Have an effective monitoring role after construction

Project aspects that affect the extent of geotechnical involvement include:

- Relative importance of geotechnical input
- Cost per unit area
- Cost as a % of total
- Feasibility and cost of overdesign
- Consequence of failure - cost and risk to life.

Table 1 illustrates different types of projects that a geotechnical engineer may be involved with. It shows the relative impact of each aspect noted above, and ranks the relative importance of the final role.

The probable role is indicated to identify those cases where the geotechnical engineer will take a lead role. Respect and effective use are characteristics that must be earned. They are illustrated by comparing the use of geotechnical engineers in natural resource development and in planning and construction in urban centres. Natural resource developments have been forced by international competition to become highly efficient. The measure of efficiency is the bottom line unit cost. To achieve new levels of efficiency natural resource developers are making increasing and effective use of the geotechnical profession.

In contrast although the geotechnical profession has much to offer toward planning and construction in the development of urban centres the services are often underutilized. Presumedly this results from the facts that urban developments can be very complex, that many professionals compete for management roles and there is no effective measure of efficiency.

Contractors are more cost conscious and are more likely to recognize and utilize geotechnical expertise than project managers. Direct work for contractors is a growth area.

In many instances, a team approach is used, with each discipline contributing its area of expertise. The team approach is enhanced by professional maturity in the outlook of the team players. It is also enhanced by liability considerations that restrict practice to one's area of expertise.

EDUCATION AND TRAINING

The extent of formal training in the geotechnical profession was summarized in the opening section.

Table II shows where the mature practitioner obtains his expertise. The data was collected in a survey of geoscience in the petroleum industry (Devenny: 1986). With the exception of in-house courses (which are seldom available in this field), it is believed applicable to the geotechnical professional also.

TABLE I

Geotechnical Role on Various Projects

	Cost Unit Area	Relative Importance	Cost of Overdesign	Consequence of Failure Cost	Consequence of Failure Loss of Life	Role Rank
Transportation Corridor:						
- Route Location	L	L	H	L-H	L	1
- Design	L	H	H	H	L	2
- Unstable Area	H	H	H	H	L	1
- Highway	L	M	H	L-H	L	3
Structures:						
- House	L	L	L	L	L	3
- High Building	H	L-M	L-M	H	H	2
- Bridge Foundation	H	H	L	H	H	1-2
Heavy Construction:						
- Dam	H	H	H	H	H	1
- Tunnels	H	H	H	H	H	1
Industrial Plant:						
- Site Development	L	M	M	M-H	L	1-3
- Foundations	H	H	H	H	L	1-2
Resource Development:						
- Open Pit Mine	M	M-H	H	H	L	1-3
- Underground Mine	H	H	H	H	L-M	1-3
- Oil Sands	H	H	H	H	L	1-2
- Northern	L	H	L	M	L	1-2
- Drill - on land	L	L	L	M	L	3
- Drill - offshore	H	H	H	H	L	1-2
Marine:						
- Dredging	L	L-H	L	L-H	L	3
- Island	L	M	H	L-H	L	1-3
Waste Disposal:						
- Municipal	L	L	L	M	L	3
- Toxic	M	M	H	H	H	1-2
- Nuclear	H	H	H	H	H	1-2

Legend: L = Low, M = Medium, H = High. 1 = Lead Role, 2 = Secondary Role, 3 = Minor Role.

TABLE II

Origins of Expertise
(Geoscience Service Personnel)

Source	Personal Experience Level 0-3 Yrs	3-10 Yrs	>10 Yrs
University	23%	10%	3%
In-House Training	17	6	9
External Training	4	2	5
On-the-Job	28	37	18
Pers. Upgrading	13	10	7
Pers. Experience	7	29	45
Seminars, etc.	5	2	6
Technical Liter.	4	4	7
TOTAL	100%	100%	100%

Table II also shows that the relative importance of university training diminishes with time. Ultimately, personal experience and on-the-job experience are most important. Contributions from formal courses, personal upgrading, seminars and technical literature are of equal importance. In the geotechnical field, with its region-specific and site-specific flavour, seminars and technical literature are probably more important than indicated.

The profession appears to be quite dependent on its technical society for continuing background "education".

It is extremely difficult for junior engineers to gain meaningful geotechnical work experience outside of a "consulting" core.

The geotechnical professional is trained to be on the lookout for many variables that can affect a project. This broad outlook is a valuable skill for senior management positions in other areas of practice.

FREE ENTERPRISE AND ECONOMIC INDICATORS

It was noted earlier that geotechnical practitioners tend to stick together when they form consulting companies. Many of the consulting firms are employee-owned, so they are centres of free enterprise.

In many fields of engineering, there is a natural tendency for older practitioners to reduce their engineering involvement and to increase their managerial duties. Table III suggests that geotechnical practitioners reverse the trend in their later years - presumably because the work is interesting. This trend is confirmed by the number of consultants who have given up senior management positions so they can return to practice.

There appears to be a direct correlation between the amount of work available for the geotechnical profession and the future economic climate. Given that the work of geotechnical engineers in site appraisals precedes construction, this correlation is not surprising.

To get a feel for the upcoming economy, it might be easier to poll key geotechnical consulting firms than to collect data from traditional sources.

Table III

Amount of Time That Geotechnical Engineers Spend on Engineering Work

		% ENGINEERING
(a)	By type of Employer	
	- Consultant:	70%
	- Government:	60%
	- Industry:	40%
(b)	By Age for Consultants	
	- Under 30:	82%
	- 31-35:	70%
	- 36-40:	55%
	- 41-50:	73%
	- Over 50:	70%

INTERACTION WITH THE CANADIAN GEOTECHNICAL SOCIETY (CGS)

Geotechnical practice is greatly influenced by local geological conditions and by local practice. Practitioners require a forum where they can meet to share case histories and extend their education. The Canadian Geotechnical Society fulfills many of those needs as illustrated by Table IV.

The results of the above questionnaire indicate that the members' communication and information needs are served by the CGS. The response to the last question indicates a high degree of member satisfaction with that service.

The profession and its society are dependent on one another. Member use of the society facilities is exceptional.

The Canadian Geotechnical Society provides linkage to other national and international societies. At the national level, CGS is a member of the Canadian Geoscience Council, which is the umbrella organization for Canadian earth scientists. CGS is also a member society of the Engineering Institute of Canada, the umbrella organization for engineering societies. Internationally, CGS is an active member of the International Society for Soil Mechanics and Foundation Engineers, the International Society of Rock Mechanics, and the International Association of Engineering Geologists.

Table IV

Member Response on Services Offered by the Canadian Geotechnical Society

Question	% Positive Response
Local Section:	
- Local meetings are held in your vicinity	84%
- You find the local meetings valuable	91%
- Percent of local mtgs. attended	50%
Conferences and Seminars:	
- You attend the annual CGS conference	84%
- You find the conference attended useful	100%
- You attended a CGS specialty symposium in the last 3 years	41%
- Specialty symposia attended were valuable	97%
Canadian Geotechnical Journal:	
- What % of articles do you read	38%
- The Journal is a valuable part of your Geotechnical background	97%
Other Publications and Activities:	
- You feel informed about CGS activities	95%
- Geotechnical news is a valuable source of information	97%
- You find other CGS publications valuable	71%
Ranking Society Services - how do you judge the relative benefits of the following CGS activities:	
- Local section activities	19.7%
- Annual CGS conference	12.5%
- Specialty symposia or seminars	8.1%
- Canadian Geotechnical Journal	34.4%
- Geotechnical News	17.6%
- EIC activities	3.4%
- Other	4.1%
TOTAL	100.0%
You Benefit from CGS Activities	95%

CLOSURE

Characteristics of the Geotechnical profession in Canada are outlined in the preceding paragraphs. Unique characteristics are identified along with the interdependence between the profession and its technical society.

Much of the data quoted in this paper was derived from a survey of CGS members (Devenny: 1985). Information on the origin of technical expertise was derived from a survey of the petroleum industry (Devenny: 1986).

Readers wishing to know more about geotechnical practice are invited to read the forthcoming book on this subject; namely, "Geotechnical Engineering in Canada", edited by Chapuis and Devenny (1988).

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Early History of the Geotechnical Profession in Canada

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SUMMARY

The journals and diaries of the early explorers, furtraders and settlers of Canada indicate that some of these pioneers had more than just a little appreciation of the importance of the ground on which they travelled and lived. These early documents include direct references to a number of geotechnical subjects -- field tests on clay, earthquakes, landslides, permafrost, tar sands and an early reference to a St. Lawrence Seaway-like canal system penetrating the continent.

As the colony developed into a country in the 1800's, the rivers, lakes and early trails gave way to canals, railways and engineered roads as the prime transportation routes. Early "military" and "civil" engineering practicians had to deal with a wide variety of geotechnical problems associated with the construction of these "new" transportation routes. Several Canadian universities began to offer engineering courses and programs. Engineering gain some acceptance with the formation of the predecessor of the Engineering Institute of Canada in 1887.

The late 1800's and early 1900's brought a natural demand for larger and more sophistocated buildings, water supplies for the quickly growing urban areas, and dams to generate hydro-electricity. Engineers oblidged and, with just the rudimentary knowledge of geotechnique, were usually successful. Along with the basics of soil mechanics, the importance of geology in the construction of these "civil" works and in the area of natural hazards, gained some acceptance.

This paper follows the early history of the geotechnical profession in Canada up to 1936 -- the year of the 1st International Conference of Soil Mechanics and Foundation Engineering. By the mid-1930's the foundations of Canadian geotechnique had been laid. Upon these foundations, the history of the geotechnical profession in Canada was prepared to be built.

RÉSUMÉ

Les journaux personnels des premiers explorateurs, pelletiers et colons du Canada, indiquent que certains de ces pionniers avaient plus qu'une petite idée de l'importance du sol sur lequel ils circulaient et vivaient. Ces premiers documents font mention d'un bon nombre de sujets géotechniques: essais in situ sur l'argile, tremblements de terre, glissements de terrain, pergélisol, sols bitumineux, et une première référence à un canal pénétrant le continent, semblable à la voie maritime du Saint-Laurent.

Dans les années 1800, alors que la colonie devenait un pays, les rivières, les lacs et les premières pistes cédèrent la place aux canaux, aux chemins de fer et aux routes comme voies principales de transport. Les premiers ingénieurs "militaires" et "civils" devaient traiter une grande variété de problèmes géotechniques liés à la construction de ces nouvelles voies. Plusieurs universités canadiennes commencèrent à offrir des cours et des programmes de génie. Le génie fut accueilli favorablement avec la création, en 1887, de la société qui devint l'Institut des Ingénieurs du Canada.

À la fin du 19^{e} siècle et au début du 20^{e}, il y eût une demande pour des édifices plus complexes et plus imposants, pour l'approvisionnement en eau des zones à urbanisation rapide, et pour des barrages hydro-électriques. Les ingénieurs rendirent ces services, et malgré des connaissances rudimentaires en géotechnique, généralement ils réussirent. On fut sensibilisé aux principes de la mécanique des sols et à l'importance de la géologie pour ces ouvrages "civils", ainsi que pour les risques naturels.

L'article décrit les débuts de la profession géotechnique au Canada jusqu'en 1936, année du premier Congrès international de mécanique des sols et travaux de fondation. Au milieu des années 30, les fondements de la géotechnique canadienne étaient en place. Sur ces fondements pouvait être bâtie l'histoire de la profession géotechnique au Canada.

This year, 1987, marks the hundredth anniversary of engineering, as an organized body, in Canada. From the viewpoint of the geotechnical profession, however, last year, 1986, was a year of note as well. Fifty years before, in 1936, the 1st International Conference on Soil Mechanics and Foundation Engineering was held at Harvard University in Cambridge, Massachusetts. This conference is considered to be the beginning of what was then called "soil mechanics and foundation engineering" and what is now referred to as "geotechnical engineering". Since 1936, the geotechnical profession has developed into a sophistocated branch of engineering that combines the knowledge of geology with the knowledge and experience of how the ground behaves naturally or how the ground will behave under the influence of changes imposed by Man.

The development of the geotechnical profession in Canada since 1936 will be the topic of several other papers in this collection. This paper traces the early history of the profession and illustrates, with a number of examples, where the fledgling Canadian geotechnical profession was at the time of the 1936 international conference, and how it got there.

The reader will note that no figures accompany this paper, except for a map of Canada that shows the locations of the major localities discussed in the text. During preparation of the text for this collection, it was realized that to properly document the early history pictorially, approximately 25 to 50 additional pages would be required. Instead of presenting a less than proper record, it was decided to forego the figures. It is hoped that at some future date, a pictorial compendium can be published.

EARLY INHABITANTS, EXPLORATIONS AND SETTLEMENTS

The first inhabitants of what is now Canada were Indians and Inuit who, it is believed, arrived from Asia as early as 40,000 years ago. Scanty evidence indicates that the first Africans and Europeans may have visited this land as recently as 1,500 years ago. About 1000 A.D. Viking arrived and settled for a short time at L'Anse aux Meadows, on present day Newfoundland. It is unlikely, however, that these early inhabitants and explorers had anything but an intuitive appreciation for the mechanical behavior of the soil and rock which surrounded them. If they had, there is no written record.

John Cabot landed on the east coast of present day Canada in 1497 and claimed the land for England. Jacques Cartier made a similar visit on behalf of France in 1534. Cabot recorded, "The soile is barren in some places, and yeeldth litle fruit [sic]". Cartier, apparently sighting Labrador, wrote, "I did not see one cartload of earth, I believe that this is the land God gave to Cain". Although inauspicious, these may be the first recorded evaluations of the Canadian terrain.

The first European settlements in Canada (the name "Canada" was alledgedly coined by Cartier from the Indian word "kanata", meaning village or community) met with varying degrees of success. Charlesbourg-Royal, a fort built by Cartier in 1541, twelve kilometres upstream from present day Quebec City, was abandoned in 1543. The settlement on Saint Croix Island, built along the present border of New Brunswick and Maine by Pierre de Monts and Samuel de Champlain, barely lasted the winter of 1604-1605. Port Royal, along the shores of the Annapolis Basin in Nova Scotia, was built by Jean de Poutrincourt and Champlain in 1605 and flourished with mild success until 1614 when it, too, was abandoned.

Geotechnique played a very small part in the location and relative success or failure of these settlements. At Port Royal, however, in 1607, Poutrincourt ordered the building of a water-driven grist mill to grind the wheat which the settlers introduced to the area and grew well nearby. This small mill with its stone foundations, wooden superstructure and undershot wooden water wheel has the distinction of being the first civil engineering structure to be built in Canada. Its construction also initiated the Canadian tradition of harnessing streams and rivers for power.

Ironically it is only a few kilometres from this site where the Tidal Power Corporation of Nova Scotia began operation of its tidal power pilot plant in the Bay of Fundy in 1983. The previous year, the Nova Scotia Power Commission completed a reconstruction of the original 1607 mill (Legget, 1982a).

A year after construction of the original mill, in 1608, Champlain founded "Habitation", which as the present day Quebec City has the distinction of being the oldest continuously occupied settlement in Canada. From Habitation, Champlain explored westward as far as Lake Huron. In 1611, "at a league's distance from Mount Royal" (now Montreal) he exhibited his inquisitive and practical nature and ordered what may be the first test of applied geology to be carried out in Canada.

> "There are also many level stretches of very good rich potter's clay suitable for brickmaking and building, which is a great convenience. I had a portion of it prepared, and built there as a wall, four feet thick, three or four feet high and ten yards long, to see how it would last during the winter when the waters came down" (from Bigger, 1925).

The results of this experiment, which were conducted on material that we now refer to as Champlain Sea marine sediments or Leda clay, are not provided in Champlain's notes.

The 1600's saw a flurry of exploration and settlement in the "New World" or "New France". Prominent among the visitors and inhabitants were the Jesuits. From their records, or "relations", which were conscientiously and painstakingly written in Latin, we find the first references to earthquakes and landslides in Canada.

> "On St. Barnabas Day (June 11, 1638) we had an earthquake in some places [near present day Trois-Rivieres, Quebec] and it was so perceptible that the savages were greatly surprised to see their bark dishes collide with each other, and water spill out from their kettle. This drew from them a loud cry of astonishment" (from Smith, 1962).

On February 5, 1663 another, stronger earthquake was recorded near Murray River (Riviere La Malbaie) northeast of Quebec City. It was accompanied by numerous landslides along the northshore of the St. Lawrence River between the Maurice and Saguenay rivers. The village of Les Eboulement (The Landslides) derives its name from this event. From such humble beginnings come the present day Seismic Zoning Maps of Canada (the most recent of which were prepared for the 1985 National Building Code of Canada) and the extensive studies of slope stability associated with the Champlain Sea marine sediments.

FURTRADERS

The 1600's and 1700's also brought the furtraders -- both English and French -- to Canada. They followed Champlain's early transportation route westward along the St. Lawrence, Ottawa, Mattawa and French rivers to Lake Huron, Lake Superior and beyond. They followed Frobisher, Davis, Hudson and Baffin northward and into Hudson Bay, James Bay and the high Arctic. The early furtrading posts were built as centres for trade with the Indians. Rivalry between the English and French and hostilities with the Indians, however, required that they also be constructed for defence from attack. Two of the largest furtrading centres were Fort Prince of Wales and the Fortress of Louisbourg.

Fort Prince of Wales is located on a peninsula at the mouth of the Churchill River on the shore of Hudson Bay. Construction of this Hudson's Bay Company fort began in 1688. Between 1731 and 1771 the English enlarged and fortified the fort with stone walls that were 9 to 12 m thick at the base and 6 m high. During these periods of construction the English encountered permanently frozen ground -- or permafrost -- for the first time. Despite the military fortifications, the English surrendered the fort to the French without a fight, when attacked in 1782.

The Fortress of Louisbourg was built by the French on Cape Breton Island (now a part of Nova Scotia) to defend the Atlantic approach to "New France". Construction of the 3 m thick, 10 m high stone walls began in 1720 and continued until 1745 when the fortress was captured by the English. It was returned to the French by treaty in 1748 and construction continued until 1758 when it was recaptured by the English and this time completely and systematically demolished.

Today Louisbourg would cost approximately $200 million to build. Part of the reason for the high cost was due to the fact that, even though the fortress was built on rock, the stone for the massive walls was shipped from France. It is also interesting to note that some of these same stones, after the fort was razed, were transported by ship to the "new" town of Halifax where they were used to construct a number of that city's original stone structures.

During the reconstruction of the Fortress of Louisbourg in the 1960's and 1970's, evidence was found to support the geological theory that sea level is rising relative to the Maritimes by approximately 30 cm per century (Grant, 1975). Among other features, the sewage and storm-water sluices which empty through a seawall are nearly one metre below present high tide. Grant (1975) dryly commented that, "If high tide had been as high during occupation of the fortress, disagreeable reversals of flow would have occurred daily".

Both Fort Prince of Wales and the Fortress of Louisbourg are now National Historic Sites, and are being preserved by Parks Canada.

To a large extent, the early furtraders were also the early explorers of the country. They were keen observers and, while being propelled by a troop of canoeists along the waterways, had time to reflect on what they saw and had time to record their thoughts and observations. Alexander Mackenzie is an excellent example of a furtrader-explorer-racounter. In 1789, at age 26, he discovered the river which today bears his name to the western Arctic. Four years later he became the first European to cross North America by land and reach the Pacific Ocean.

On his way to Fort Chipewyan, on the shores of Lake Athabasca in northern present day Alberta and from where he began both his epic "voyages", he descended the Athabasca River. Although not the first European to see the Athabasca tar sands, he was the first to record their presence.

> "At about twenty-four miles from the Fork [downstream from present day Fort McMurray, Alberta], are some bitumenous fountains, into which a pole of twenty feet long may be inserted without the least resistance. The bitumen is in a fluid state, and when mixed with gum, or the resinous substance collected from the spruce fir, serves to gum the canoes. In its heated state it emits a smell like that of sea coal. The banks of the river, which are there very elevated, discover veins of the same bitumenous quality [sic]" (from Lamb, 1970).

During his 1789 trip down the Mackenzie River he noted burning coal seams (which are still smouldering today) just upstream of Fort Norman. "In other places the bank of the river is high of black earth and sand continually tumbling, in some part shews [sic] a face of solid ice, to within a foot of the surface" (from Lamb, 1970). The coal and permafrost to which Mackenzie briefly referred, have become major fields of geotechnical study and research in Canada, especially in the past two decades. The recent development of the Athabasca Tar Sands was just this year selected as one of Canada's ten outstanding engineering achievements of the past 100 years.

Mackenzie was also farsighted. In 1801 he penned a letter to Lord Hobart, Secretary of the Colonies in the British Government entitled "Memorandum concerning a canal projected by the American States from Albany to Lake Ontario; and a Canal between Lake Ontario and Montreal, by which the former would be rendered fruitless." (Lamb, 1970). In short, he was advocating a forerunner to the St. Lawrence Seaway -- a project that would give Canada a trading advantage over the Americans and their soon-to-be-built Erie Canal. From a geotechnical viewpoint Mackenzie noted that along his proposed route "the ground is well adapted to cutting of such passage and water can never be wanting as long as there is any in the lakes of Canada". At that time, his proposal was developed no further, however, during the middle of the present century the St. Lawrence Seaway was constructed jointly by Canada and the United States, and it too has recently been selected as one of the ten outstanding engineering achievements of the past century.

AGE OF CANALS

In Canada up to the early 1800's, the primary transportation routes were the rivers and lakes. Any roads at that time were essentially simple horse trails and/or portages. It is not surprising, therefore, that the early men of commerce proposed canals to improve the water ways.

The earliest excavation for a canal began in 1680. The Lachine Canal was created to bypass the Lachine Rapids on the St. Lawrence River, but this initial, hand dug canal was only large enough to handle canoes. So important was this route, that the canal was enlarged and/or modified in the 1700's, 1800's and 1900's. It served as a most important transportation route until 1959 when the St. Lawrence Seaway was opened (Legget, 1968, 1974).

The first dredging operation in Canada took place in 1747. It was associated with deepening the Richelieu River near St. Ours, Quebec, thereby improving the water transportation link between Lake Champlain and the St. Lawrence River. In 1797 the North West Company, rival of the Hudson's Bay Company, built the first lock in Canada at Sault Ste. Marie, Ontario. This small wooden lock greatly assisted the "York" boats in travelling from Lake Huron to Lake Superior.

Numerous other canals were proposed for the Maritimes, and Upper and Lower Canada, but it was not until after the War of 1812-1814, between British North America and the United States, that other canals were constructed. For purposes of defence. a water route, alternate to that of the St. Lawrence River between Montreal and the east end of Lake Ontario, was proposed. It would follow the Ottawa River up to present day Ottawa, then head cross-country by means of a canal that would generally follow the Rideau River, Rideau Lakes and the Cataraqui River to Kingston on Lake Ontario. This route required a series of locks and canals on both portions of the route.

The Grenville Canal on the Ottawa River was first surveyed in 1816 and constructed between 1819 and 1831 by the Royal Staff Corps. It was the first public work of any size to be constructed in Canada and involved the excavation of 400,000 m of solid Precambrian bedrock (Legget, 1980a). The rock was hand-excavated after blast holes were hand-drilled into the rock. This may possibly be the first drilling operation undertaken in Canada. Lieutenant Colonel Henry du Vernet, the commander of the Royal Staff Corps construction unit, knew enough about geology to leave us some of the earliest records of rock formations in eastern Canada. Du Vernet used his knowledge of "engineering geology" to locate an associated second canal on the Ottawa River at Carillon. "In order to minimize rock excavation ...he followed the lie of the land around the Carillon Rapids even though this involved locking boats up to the main part of the canal on their downstream journey" (Legget, 1976).

The Rideau Canal, which connects Ottawa and Kingston, was located and constructed between 1826 and 1832 by the Corps of Royal Engineers under the direction of Lieutenant Colonel John By. This canal, approximately 200 km in length, includes 47 masonry locks and 50 dams. It has been called one of the greatest construction achievements in North America. Besides the cunning use of the terrain, which By used to his advantage to reduce the length of artificial channels required, there are three locations that are particularly interesting from a geotechnical viewpoint.

The dam at Jones Falls is 106 m long between abutments and 19 m high, and when completed in the 1830's was the highest dam in North America and the third highest dam in the world! It was constructed by John Redpath (who later became a successful businessman in the sugar industry) as a keywork stone arch dam. Founded on bedrock, the stonework is 8.5 m thick at its base and 6.5 m at the top. A "clay and earth" apron was placed on the upstream face of the stonework at a 2.5:1 slope. A 1.5 m thick clay core was placed between the apron and stonework (Passfield, 1982). The dam was built in stages so that the earth apron with temporary spillways could act as cofferdam while the stonework was being constructed.

The proposed dam at Hog's Back, on the outskirts of present day Ottawa, was to be 52 m long and 14 m high, and of a masonry arch design. Construction began in July 1827 but when the final gap was filled in February 1828, a sudden flood washed away a good portion of the earthfill. It took a year to rebuild the washed out section, just in time for a March 1829 flood to wash away much of the dam as well as material from the west abutment. Colonel By abandoned plans for a masonry dam and, after perservering for two more years, completed a 75 m long timber crib dam (Passfield, 1982). Associated with the initial 1827 dam construction, a short railway was built to transport stone to the dam site from a nearby quarry. Although temporary, this may have been the first rail line built in Canada (Legget, 1955).

The Lower Brewers lock, near the Kingston end of the canal was built, by necessity of alignment, on a thick stratum of soil instead of bedrock, as was the accepted practice (Anon., 1978). Although construction of the heavy lock was no more difficult than normal, the stone structure underwent considerable differential settlement. The resulting problems were not fully resolved until the lock was completely reconstructed in 1976. This is possibly Canada's first and longest settlement case history.

As a tribute to Colonel By and the Corps of Royal Engineers, the Rideau Canal celebrated its 150th anniversary in 1982, is still in full operation and is recognized as a National Historic Site.

At the same time as the Ottawa-Rideau route was being constructed, the Welland Canal was being excavated to bypass Niagara Falls and thus provide a continuous water transportation route between Lake Ontario and Lake Erie. Unlike the former public work, however, the Welland Canal was built and owned by private interests. Forty locks were originally required to connect the two lakes and a one way trip usually took two days. The initial design called for a tunnel to be driven through a height of land between Lake Erie and the Niagara escarpment. This design was subsequently changed to a "deep cut". During excavation of the deep cut, geological problems were encountered which were described as "landslides". However, recent work in this area indicates that ground failure was probably due to the fact that the excavation was advanced through a thick stratum of clay and glacial till overlying more permeable sediments within which the groundwater is under sub-artesian pressure (Legget, 1980). The depth of the deep cut was reduced accordingly and two locks were constructed to lock up boats on their way down to Lake Ontario (Legget, 1976).

The Welland Canal remained private until 1841. In that year, during the first session of the Legislature of the United Province of Canada (today's Ontario and Quebec), the government assumed control of the canal under the newly created Board of Commissioners of Public Works. This Board was the forerunner of the present day Canada Department of Public Works, of which more will be said later.

AGE OF RAILWAYS

As the Ottawa, Rideau and Welland canals were being completed, the advantages of land transportation by means of railways were being studied. As we shall see, the age of canals did not die, but for the moment, Canada entered the age of railways.

The first permanent railway in Canada, the Champlain and St. Lawrence Railroad, was completed in 1836. It connected Laprairie, on the south shore of the St. Lawrence River across from Montreal, with St. John's, Quebec, on the Richelieu River. This 25 km long route shortened by 150 km the water route between Montreal and the United States via the Richelieu River, Lake Champlain and the Hudson River. It also bypassed a required portage -- and later canal and locks -- on the Richelieu River at Chambly, Quebec (Legget, 1973). Geotechnically the construction was relatively simple. Only one low but long embankment and four short and one 122 m long bridge were required.

Several other short railways, also in Quebec (or Lower Canada as it was then called) followed in the 1840's and 1850's. Upper Canada, not to be outdone, also began to "think railways". One of the first railways in Ontario was built in the early 1850's and ran between Toronto, on Lake Ontario, and Aurora, south of Lake Simcoe. Although from a geotechnical viewpoint there is little of interest connected with this route, it is significant in that a young engineer, Thomas Roy, was in charge of and carried out the first survey for this railway in the 1830's (Legget, 1980a).

Little is known of Thomas Roy. From what we do know, however, he appears, by having an understanding of the geological implications of all his engineering work, to be the first Canadian civil engineering geologist. Besides the railway, he carried out a study on the improvement of Toronto's harbour, studied navigation on the upper Ottawa River, postulated a theory for the raised beaches around Lake Ontario, prepared a geological stratigraphic section for a portion of southern Ontario, and wrote a pamphlet on "the Principles and Practice of Road-building as applicable to Canada". Published in 1841, this pamphlet predicted that railways would one day be rivalled by the "steam-carriage upon common roads". He also wrote:

> "Drainage is an affair of primary importance in roadmaking, and requires much skill to execute in a proper manner...it may even be necessary to carry the process of draining far beyond the area of the road" (from Legget, 1980a).

These same words should be heeded by today's engineers.

As the small railways proved their worth, schemes for longer railways were dreamed up, and a few were actually financed and built. The first major railway to be completed was the Grand Trunk. Built between 1845 and 1861, in sections between Sarnia, Ontario in the west and Riviere de Loup, Quebec in the east, it was -- and still is -- the main railway line for Canada. The short tunnel near Brockville, Ontario and the Victoria Bridge over the St. Lawrence are two structures that have interesting geotechnical features.

To provide rail access to the St. Lawrence River from the main Grand Trunk Railway near Brockville, a short branch line was constructed. To overcome a steep bluff, a 500 m long tunnel was excavated. Completed in 1854 this tunnel, which was used up until recently, has the distinction of being the first tunnel constructed in Canada, and the first railway tunnel in North America (Legget, 1973 and 1980a).

The St. Lawrence River was a major obstacle in linking Grand Trunk's eastern and western lines. This was solved by the construction of the 2,800 m long Victoria Jubilee Bridge at Montreal. It was constructed on masonry piers spaced approximately 75 m apart. The piers, constructed within wooden cofferdams, were founded on solid rock on the river bottom. Rock for the piers was transported to the site from Pointe Claire, to the north (using the Lachine Canal), and from Lake Champlain, to the south (using the Champlain and St. Lawrence Railroad). Construction began in 1854 and was completed in 1859, in time for the Prince of Wales to open it officially during the first Royal visit to Canada in 1860. Although the superstructure has been replaced, the original piers are still performing well (Legget, 1975).

One reason for the success of the bridge foundations is possibly the assistance provided to the design engineer, Robert Stephenson, by William Logan, geologist with the Geological Survey of Canada. In his report Stephenson related:

"I have read and studied with great pleasure an admirable and most graphic description by Mr. Logan of the whole of the varied conditions of the river..." (Legget, 1982b).

Although not usually noted for its impact on engineering, the Geological Survey of Canada has, as this example shows and as we shall see later, contributed a great deal to geotechnique in Canada.

The Intercolonial Railway was proposed to connect the Grand Trunk Railway at Riviere du Loup, Quebec with Halifax, Nova Scotia. It was completed in stages between 1858 and 1876. Of the major civil engineering works along the route, the bridges over the Miramichi River, in New Brunswick, are most interesting from a geotechnical viewpoint.

The engineer in charge was Sandford Fleming. He requested borings along the centrelines of the two proposed bridges and designed the foundations for the bridge piers accordingly -- timber caissons to be filled with tremie concrete. At one bridge site, when construction began in 1869, what was thought to be bedrock turned out to be sand and gravel underlain by silt and clay. Fleming halted construction, ordered additional, more sophistocated borings and even carried out static penetration tests to determine the bearing capacity of the underlining strata. These are likely the first penetration tests ever performed in Canada, and perhaps anywhere in the world (Legget and Peckover, 1973). He then increased the number of bridge piers from five to six and increased the loading area of the open timber caissons. During construction, Fleming noted settlement and halted construction once more so that each pier could be preloaded to encourage settlement to occur before the superstructure was constructed. Both bridges, completed in 1875, are functioning well today (Peckover and Legget, 1973).

During his tenure with the Intercolonial Railway, Fleming was asked to carry out the preliminary location survey for the Canadian Pacific Railway to the Pacific Coast of Canada. Between July 1 and October 11, 1872, Fleming, his 16-year-old son and five others travelled 8,500 km between Halifax, Nova Scotia and Victoria, British Columbia. The epic journey consisted of 1,530 km by rail, 3,495 by horse, 2,700 by steamer and 775 by canoe. Detailed records were kept by the secretary of the party, George M. Grant (Grant, 1873).

Because of political and financial reasons construction of the CPR was delayed until the early 1880's. Geologically and geotechnically the legacy of the CPR included: finding the wealth of mineralization at what is now Sudbury, Ontario; finding coal in Alberta and British Columbia; learning in a primitive way how to deal with muskeg in Northern Ontario; and learning about rock slope stability along the Fraser Canyon, landslides in glaciolacustine silt along the Thompson River and snow avalanches in Rogers Pass, all in British Columbia. Many engineers and thousands of labourers rose to the challenges, and by 1885 the Canadian Pacific Railway was complete and one could travel from coast to coast by rail. Although the right-of-way had been cleared and the tracks had been laid, geotechnical problems continued to cause the CPR to be an ongoing challange to future generations of engineers.

SOME ENGINEERED ROADS

Before the railways had a chance to open-up some of the less accessible parts of Canada, land transportation was required to assist in the development, settlement and defence of the country. Surveying and construction of early roads was the answer. Because of their primitive nature, little conscientious geotechnical foresight was applied to these roads, however, two examples will show what the early engineers were up against.

The Cariboo Wagon Road connected Yale, the upstream point of navigation on the Fraser River, British Columbia, with Barkerville, the famous British Columbia gold mining town. The building of this road was commissioned, in 1861, to simplify travel to the goldfields and open-up the undeveloped interior of the "colony" of British Columbia. The road was surveyed and constructed between 1862 and 1865 under the supervision of Colonel R.C. Moody and a Corps of Royal Engineers. The road, which is almost 800 km in length, was built to a standard 18 foot width and cost an average of $1,250/km. Approximately two-fifths of the route followed the treacherous Fraser and Thompson river canyons where in many places it had to be literally notched into the side of, or in part suspended over, the canyons. The construction of the original Trutch Bridge, the first bridge over the Fraser River, and the continual problem with Drynoch Landslide are only two items of engineering interest associated with this early road (Sanderson, 1976; and VanDine, 1983). Two railways and the Trans-Canada Highway follow portions of the original route today, and today's engineers still are faced with geological, geotechnical and engineering challenges similar to those of their predecessors.

The Department of Public Works saw the need of a transportation route between Lake Superior in northwestern Ontario and Selkirk on the Red River in Manitoba. The route was first surveyed by Simon Dawson in 1859, but it was not until 1868 that he was awarded the contract to begin construction. The route was just under 700 km in length and consisted of an 80 km stretch of road between Fort William (present day Thunder Bay) and Lake Shebandowan, a 170 km stretch of road between Lake of the Woods and Selkirk (near present day Winnipeg) and approximately 450 km of water travel, with up to 70 portages, inbetween (Legget, 1974). It was completed in 1870. To think of the difficulties the CPR had in crossing similar terrain immediately to the north, ten years later, the construction of the early engineered road segments and portages is a credit to Dawson and Canadian engineering at that time. It is interesting to note that it took Fleming's 1872 CPR survey party 10 days to cross the Dawson Road whereas previously it might have taken a traveller 3 to 4 weeks to traverse the same route (Grant, 1873).

EDUCATION AND ORGANIZATION

The second half of the 1800's was a period of expansion and development for the fledgling country of Canada, and this was reflected in the growth of engineering. One of the ways in which engineering progressed was the introduction of civil engineering into university programs: at the University of New Brunswick in 1854; Toronto in 1859; McGill in 1871; Ecole Polytechnique in 1873; Royal Military College in 1876; and Queen's in 1893. The early thrust of these civil engineering programs was, not surprisingly, surveying and construction -- especially related to transportation in the form of canals, railways and roads (Richardson, 1976b).

In 1887, the forerunner to the Engineering Institute of Canada -- the Canadian Society of Civil Engineers -- was formed. The first President, Thomas C. Keefer had worked on the Welland Canal, oversaw construction of a portion of the Grand Trunk Railway and later became one of the few Canadians to be President of the American Society of Civil Engineers (Richardson, 1976a).

Samuel Fortier, an 1885 civil engineering graduate of McGill University, and a member of the newly formed Canadian Society of Civil Engineers, made his mark on geotechnique, however, in the United States as opposed to Canada. He worked for the U.S. Department of Agriculture, eventually rising to the head of the irrigation section. In 1896 he presented a paper to the Canadian Society of Civil Engineers which dealt with the design and construction of earth dams. He stressed the importance of knowing the properties of the soils with which an earth dam was to be built, and described the process of soil compaction. Ahead of his time, Fortier stated:

> "For twenty years and over men have been testing the physical qualities of iron, steel, cements...Reservoir embankments on the other hand have been built in most instances without the requisite knowledge, upon mere guess work, brawn and not brain predominating" (from Legget, 1980a).

MORE CANALS AND RAILWAYS

Towards the end of the 1800's and into the beginning of the 1900's, civil engineering was becoming more sophisticated and engineers were developing more of an appreciation for proper investigation of the ground conditions prior to construction. In this regard, several engineering projects of that era, which were associated with canals and railways, provide excellent examples.

The Murray Canal, which provides Trenton, Ontario and the Trent Canal system easy access to Lake Ontario, was first suggested in 1796. The 8 km canal, which has no locks, was finally constructed between 1882 and 1889. Over the intervening years, two proposed routes were suggested and studied: the present route and a much shorter route. During 1880 and 1881, 13 test pits and 500 test borings were made along the proposed routes, and based upon the finding of this subsurface investigation the longer route was finally selected. Along the shorter route, most of the excavation would have had to be in rock, while there was no rock excavation along the longer route (Legget, 1980b).

To connect the Grand Trunk Railway, which served Quebec and Ontario, with Chicago and the mid-western United States, a railway tunnel was proposed under the St. Clair River at Sarnia. Construction of the tunnel began in 1889 and was completed in 1891. It was the first underwater railway tunnel to be built in North America! Of interest to the heritage of geotechnique, however, is the fact that prior to final location and construction, test borings were advanced on 6 m centres across the St. Clair River and along the approach cut locations. Glacial clay with pockets of wet sand and some boulders were encountered, and the engineer-in-charge, Joseph Hobson, designed the tunnel and the tunnelling procedures accordingly. Two steel "hydraulic travelling shields", one starting at each end of the tunnel, were used along with compressed air to complete the 1.8 km long tunnel. The tunnel is still in service today (Legget, 1979).

In 1615, Champlain used the Ottawa River, Mattawa River, Lake Nipissing and the French River to gain access to Lake Huron. This route, known as the Ottawa Waterway, remained the major furtrade route for the next two hundred years. In 1827, Colonel By, obviously enthusiastic about canal building, suggested the construction of a canal along the entire Ottawa Waterway. This route was approximately 500 km shorter than the Lake Ontario, Lake Erie, Lake St. Clair route -- its rival. During the 1800's the idea was revived several times and between 1856 and 1859 two surveys were carried out, but nothing was done. In the late 1800's and early 1900's, the idea of what was renamed the Georgian Bay Ship Canal was raised again. Geologists from the Geological Survey of Canada mapped the geology of route and the Department of Public Works spent almost one-half million dollars on a thorough site investigation. This extensive subsurface investigation included 2,990 test boring advanced with a churn drill at the proposed dams, locks and along the route. The final report estimated construction costs would be $100 million, annual maintenance costs would be $900,000 (both in 1909 dollars) and that it would take 10 years to build (Legget 1976 and 1980a). The canal has yet to be built although as recently as the 1960's the idea once again became popular.

In the early 1900's railway engineers were once again challenging nature. A second railway, the present Canadian National Railways, was completed across the continent and many shorter but more difficult rail lines were located, surveyed and built.

In 1909 the CPR reduced the grade on its mainline by completing the two spiral tunnels through the Kicking Horse Pass in British Columbia. Between 1913 and 1916 they constructed the 8 km long Connaught Tunnel through Rogers Pass. This tunnel will have been the longest railway tunnel in North America until the completion of the 15 km long new Rogers Pass tunnel which is presently under construction in the same area. (This new tunnel will be used for CPR westbound trains, while the Connaught Tunnel will be kept in service for the eastbound trains.) The Connaught Tunnel not only reduced the grade of CPR's track to 2.2%, but also bypassed the worst snow avalanche area in Canada. Very little geotechnical work was carried out in association with these early 1900 railway tunnels, but they must be included to show the sophistication of civil engineering works being carried out at that time (Dennis, 1917).

Between 1910 and 1929 the Hudson Bay Railway, one of the Canadian National Railways, was built between La Pas, and Churchill, Manitoba (very close to the site of Fort Prince of Wales). This 800 km long railway is important for two geotechnical reasons. This may be the first Canadian civil engineering work of any size to be constructed on discontinuous and continuous permafrost. Observations as to the distribution of the permanently frozen ground, its effect on the construction and maintenance of the railway, and how the construction problems were solved, were duly noted by the location and construction engineers (Charles, 1959).

The Hudson Bay Railway may also be the first major transportation project for which air photographs were used for route alignment and used to predict geological and engineering purposes -- a forerunner to engineering terrain analysis:

> "Unable to decide whether Nelson or Churchill would make the best port on Hudson Bay, the federal government in the summer of 1925 ordered that the whole route to Port Nelson and the region surrounding it be photographed from the air to augment a study of the terrain by engineers" (Thomson, 1975).

The government's decision to use air photographs was made shortly after the first paper on airphoto interpretation was published in the Journal of the Engineering Institute of Canada, "The Use of the Aeroplane in Surveying and Engineering" (Vol. VII, No. 1, 1924 - Wilson, 1924). Today airphotos are taken for granted on all large engineering projects, and Canadians are noted as leaders in the field of airphoto interpretation for engineering purposes.

With Canada's extensive legacy of railway location and construction, it is not surprising that Canada's railway network was selected as one of Canada's ten outstanding engineering achievements over the past 100 years.

FOUNDATIONS

The early settlers of Canada constructed buildings on all types of foundations and on all types of foundation conditions. Many of the buildings were small, the foundations were suitable and the foundation conditions, fortunately, were adequate. Undoubtedly there were some failures, but the record of these, for the most part, is missing. One case history, however, of a mid-1800's partial foundation failure has been documented.

Christ Church, the Cathedral Church of the Anglican Diocese of Montreal, was constructed under two contracts between 1856 and 1859. The first contract was for the foundations; the second for the superstructure. Before the second contractor finished the stone building with its 70 m high spire, he noticed that the foundation was settling differentially. When completed, the tilt of the steeple was 12 cm off vertical and the church stopped all further payments. In what may be the first piece of geotechnical litigation in Canada, the contractor took his claim for payment to court. He lost his case in the Canadian court and appealed it to the Privy Council in London. That body dismissed his case as well. The findings in this landmark case were:

> "If the builder thinks fit to trust to the vigilance or skill of the architect, without the independent exercise of his own judgement, he acts on his own risk. He cannot escape from liability..." (from Legget, 1977).

By 1927, settlement of the church reached 16.5 cm and since the steeple was precariously 60 cm out of plump, it was removed. A recent reconstruction of the foundations, this time after a soils investigation, has allowed the steeple to be replaced.

In the early 1900's, the knowledge of how soils behave under loading conditions was still in its infancy. The size (and therefore the weight) of the structures and buildings, however, were increasing. This situation led to numerous difficulties with foundations. Three of Canada's best documented foundation problems in the early 1900's are the Empress Hotel in Victoria, B.C., the National Museum Building in Ottawa, Ontario and the Transcona grain elevator near Winnipeg, Manitoba (Crawford and Sutherland, 1971; Crawford, 1953; Baracos, 1957).

The Empress Hotel evidenced differential settlement due to the combined load of a fill and the structure over a varying thickness of soft marine clay over bedrock. This building has the distinction of having the longest continuous settlement record in the country. The National Museum was constructed on 30 m of sensitive Champlain Sea marine clay and the differential settlement was caused by differential loading in areas of the building. Maximum settlement was 30 cm. The Transcona grain elevators is a very large grain holding structure. When it was initially filled, a classic bearing capacity failure of the clay soil occurred that left the elevators at an angle of 27 degrees off plump. In all three cases, remedial measures were undertaken and all three structures are still in active service today.

In 1918, the Canadian Society of Civil Engineers changed its name to the more appropriate Engineering Institute of Canada. At the same time the Engineering Journal, the journal of the new institute commenced publication. The first paper in that journal to be concerned with foundations and geotechnique appeared in Volume IV, 1921 and was contributed by D.S. Ellis. In that paper he reviewed an early 18th century textbook by Belidor, a French professor of mathematics. Belidor's text, based to a great extent on the work of de Vauban, the great engineer of Louis XIV, was published in 1739 and entitled "La Science des Ingenieurs dans la Conduite des Travaux de Fortification et d'Architecture Civile". Among other items, Ellis described theories of retaining walls, earth pressures against retaining walls, and foundations. Just slightly ahead of their time, Belidor and de Vauban were quite knowledgeable about foundations on rock, competent soil, and soft soil; pile foundations; consolidation; the importance of groundwater; and the use of cofferdams for underwater foundations. And this in 1739, and described in 1921!

The first bona fide geotechnical paper in the Engineering Journal appeared in Volume IX, 1926. A. W. Fosness, a design engineer with Carter-Halls Aldinger Co. of Winnipeg, Manitoba described the foundation conditions in the Winnipeg District. He discussed floating and pile foundations; allowable bearing; causes of settlement; foundations on clay at different elevation along river banks; the influence of frost and the relative cost of various foundations.

WATER RESOURCES

Water supply in Canada, up to the early 1900's, was usually fairly straight forward because of the large number of fresh water lakes close to the urban areas. After 1900, however, the exceptional growth of some of the larger centres required city engineers to look for new water supplies. One of the earliest and most ambitious projects was to carry water by means of a 160 km long aqueduct from northwestern Ontario to the City of Winnipeg, Manitoba. The foundations for the closed horseshoe shaped concrete aqueduct, constructed between 1913 and 1919, varied depending upon the grade and foundation conditions encountered over its length (Fuertes, 1920 and Chase, 1920). The engineers accepted the challenge with the result that there were no major foundation problems during construction and there have been none since. It is interesting to note, however, that just after construction was complete, degradation of the concrete footings and buried sections of the aqueduct was noticed. The resulting investigation found that the degradation was due to the sulphate in the ground. A successful research program by Dr. T. Thorvaldson of the University of Saskatchewan led to the development of the first sulphate resistant cements, later commercially promoted by the Canada Cement Company (Bubbis and Sommerville, 1960).

Besides water supply, water power was becoming more important to Canada in the late 1800's and early 1900's. The first electric dynamo was installed at Chandiere Falls on the Ottawa River in 1882. In 1893, two 1,000 hp units were installed at Niagara Falls (Richardson, 1976a). The Canadian tradition of hydro-electricity, in which Canadian engineers would become the accepted world leaders in the 20th century, had begun.

All across Canada in the early 1900's hydro-electric dams were being planned, designed and constructed. In 1926 the first paper in the Engineering Journal dealing with a hydro-electric dam site was published. E.E. Carpenter, a consulting engineer for the British Columbia Electric Railway Company, described the in-progress construction of the ambitious Bridge River Project in the Coast Mountains of British Columbia, northeast of Vancouver (Carpenter, 1926). In his paper he described what we would now call the geotechnical subsurface investigation to find suitable locations for the diversion and storage dams. Test boring were advanced, permeability tests were conducted and estimates of the bearing capacity were made. Subsequently two diversion dams were constructed and a 4,000 m long, 3.5 m diameter tunnel was advanced through Mission Mountain to the power station on Seton Lake.

Several other early hydro-electric developments are worthy of note from a geotechnical viewpoint. In 1929, a 25-year old civil engineer from Liverpool, England, arrived in Canada and took up the position of Resident Engineer for the Power Corporation at the proposed Upper Notch hydro-electric plant on the Montreal River in Northern Ontario. This was Robert F. Legget's first job in Canada.

In 1930, Calgary Power completed its Ghost Power hydro-electric development on the Bow River, 50 km west of Calgary, Alberta. The dam is 1,500 m long and has a maximum head of 33 m. The dam was designed and constructed as two earth fills on either side of a central concrete section. Of interest geotechnically is the fact that the earth fills, one 225 m long and the other 620 m long, were constructed as hydraulic fills. Their design and construction are described fully in a 1930 Engineering Journal paper by W.H. Abbott of the Foundation Company of Canada.

The Beauharnois, Quebec hydro-electric development, just west of Montreal, was constructed between 1929 and 1932. It was a significant geotechnical undertaking because so much of the site was underlain by sensitive Champlain Sea marine sediments (Leda clay). Detailed subsurface investigations were carried out to delineating the location of clay, till and bedrock. Clay samples were tested in the laboratory for unit weight and water content. D.W. McLaughlin, an experienced railway and canal engineer employed by the Department of Public Works, suggested "insitu" shear tests be conducted on the clay and he devised a primitive test apparatus:

> "The device was a four-finned tool (diameter about 6" and length maybe a foot) pushed down into the Leda clay, and used to determine shear strength by measuring the twisting torque" (from Burpee, pers. comm.).

McLaughlin had discovered the "vane shear test"! Fifteen years later, during a visit to the Beauharnois site in 1944 Terzaghi demonstrated his new idea -- that of taking "undisturbed" Leda clay samples for laboratory testing.

GEOLOGY AND GEOTECHNIQUE

The Geological Survey of Canada (GSC) was founded in 1842 and William Logan (later Sir William) was appointed the first Director. It is perhaps ironic that the first paper that he published after he arrived in Canada presented the results of some of his early research on landslides in the Champlain Sea marine sediments along the St. Lawrence River valley (Legget, 1980a). As mentioned previously, Logan went on to contribute significantly to a number of engineering projects in the country, for example, the Grand Trunk Railway's Victoria Jubilee Bridge near Montreal.

During the latter half of the 1800's, only minor progress was made to include more geology into the design and construction of engineering work. In 1873, the GSC purchased the first steam-driven diamond drill in Canada. This was used in the 1870's and 1880's to locate various construction materials for the construction of the CPR and, as Selwyn, then Director of the Geological Survey, stated to "extend and hasten the exploration and survey in the North West Territory" (Scott, 1979).

In 1900 Henri Ami, a geologist and palaenontologist with the GSC published a Royal Society of Canada paper entitled "On the geology of the principal cities in Eastern Ontario". This paper summarized the geological formations underlying Saint John, Quebec, Montreal, Ottawa and Toronto (Ami, 1900). In 1902, Ami undertook, on behalf of the Department of Railways and Canals, a geological examination of the materials encountered in borings for the abutments and piers for the, now famous, Quebec Bridge (Ami, 1903).

Landslides continued to interest geologists. R.S. Ells, in 1904, published the results of his investigation of recent landslides in the Ottawa Valley-St.Lawrence Lowlands (Ells, 1904). The large rockslide at Frank, Alberta in 1903 was studied by R.G. McConnell and R.W. Brock, and until the 1970's their report of 1904 was used as the classic example of a joint plane failure in rock (McConnell and Brock, 1904).

Geologists also became involved with Canada's early 1900's transportation network. As the automobile became more prominent, GSC staff were employed to find better road building materials in Ontario and Quebec (Blais et al, 1971). When the Department of Public Works wanted to improve navigation on the Fraser River delta south of Vancouver, in 1919, they requested W.A. Johnson, another geologist with the GSC, to carry out a geological investigation and engineering assessment of the area. In 1926, the Department of Railways and Canals began improvements on the historic Lachine Canal. The Geological Survey was asked to interpret the results of borings associated with the engineering investigations for lock construction (Scott, 1979).

At the University of Alberta in Edmonton, in 1925, Professor John Allen of the Department of Geology began a geology course specifically designed for civil engineering students. This is possibly the first engineering geology course to be offered in Canada. Allan was not only an academic, but a practicing engineering geologist as well. In 1927 he published a paper in the Engineering Journal entitled, "Geological Aspects of the Spray Lake Water Power Project". This is the first engineering geology paper to appear in the journal (Allen, 1927).

In 1921 Professor H. Ries of Cornell University and T.L. Watson of the University of Virginia published the first engineering geology text (Ries and Watson, 1921). Eight years later, in 1929, Ries published a paper in the Journal of the Engineering Institute of Canada entitled, "The Importance of Geology to Civil Engineering" (Ries, 1929). In a discussion of Professor Ries' paper Professor Allan foresightedly commented:

> "The day was coming...when the practicing civil engineer would invariably have the geological problems associated with the particular project on hand investigated by one qualified in that profession"(Allen,1929).

This was the theme to be taken up by that young civil engineer, Robert Legget. By the early 1930's Legget was convinced of the importance of geology in civil engineering and with the encouragement of Professor O'Neill of the Department of Geology at McGill he wrote his first paper for the Engineering Journal, "Geology and Civil Engineer: Their Relationship, with Reference to Canada" (Legget, 1934). From this 12 page article grew Legget's first textbook entitled "Geology and Engineering" published in 1939. In 1983, what is essentially the third updated edition of this same text, was published under the title "Handbook of Geology in Civil Engineering" (Legget and Karrow, 1983).

...AND ON TO 1936

This was the state of geotechnical affairs as Canada entered the depression years of the 1930's. The early explorers, settlers and furtraders had come and left their mark. Canals and railways had, in turn, had their day and now roads were becoming more popular. Development of northern Canada was beginning. Buildings were increasing in size and weight and required more sophistocated foundation engineering. Canada was in the forefront of developing its water resources into water reservoirs and hydro-electricity. Geology was recognized as being of some importance to civil engineering.

In the spring of 1936, a small item appeared in an issue of the United States engineering publication, "Engineering News Record", about a proposed conference on soils and soil mechanics to be held at Harvard University, June 22 to June 26 of that year. The President of the conference was to be Dr. Karl Terzaghi, author of the 1925 textbook entitled "Erdbaumechanick (Soil Mechanics)", and a visiting professor at Harvard University from the Technische Hochschule in Vienna, Austria. The conference attracted the attention of approximately 200 engineers from all over the world including eight from Canada: from the Department of Public Works, H.M. Davy and Jack Lucas of the test-drilling and testing laboratories branches respectively; C.R. Young from the University of Toronto, I.F. Morrison from the University of Alberta and G.M. Williams from the University of Saskatchewan; and from industry R.J. Mattson of Foundation Company of Canada, J.M.R. Fairbairn, Chief Engineer of the CPR and R.F. Legget at that time employed by the Canada Steel Piling Company (Legget, 1936).

This conference marked the birth of geotechnique under the name of soil mechanics and foundation engineering. Not only was Canada well represented at the birth, but as we hope this review of the early history of the geotechnical profession in Canada shows, Canadians were also active in the gestation period.

ACKNOWLEDGEMENTS

As a glance through the references might indicate, the author must first thank Dr. Legget for recording many of the little known facts about engineering history in Canada. Not only have his references been a treasure house of information, but over the years they have acted as a catalyst to the author to delve into Canadian history searching for others unknown references to early engineering in this country. In addition, Dr. Legget has spent many hours reviewing and improving various drafts of this manuscript. The author would also like to thank all others who assisted in the research and preparation of this paper. As you might imagine the list is very long. Dr. J.I. Clark, Mr. D. Townsend and Dr. D.Devenny deserve the credit for instigating its preparation.

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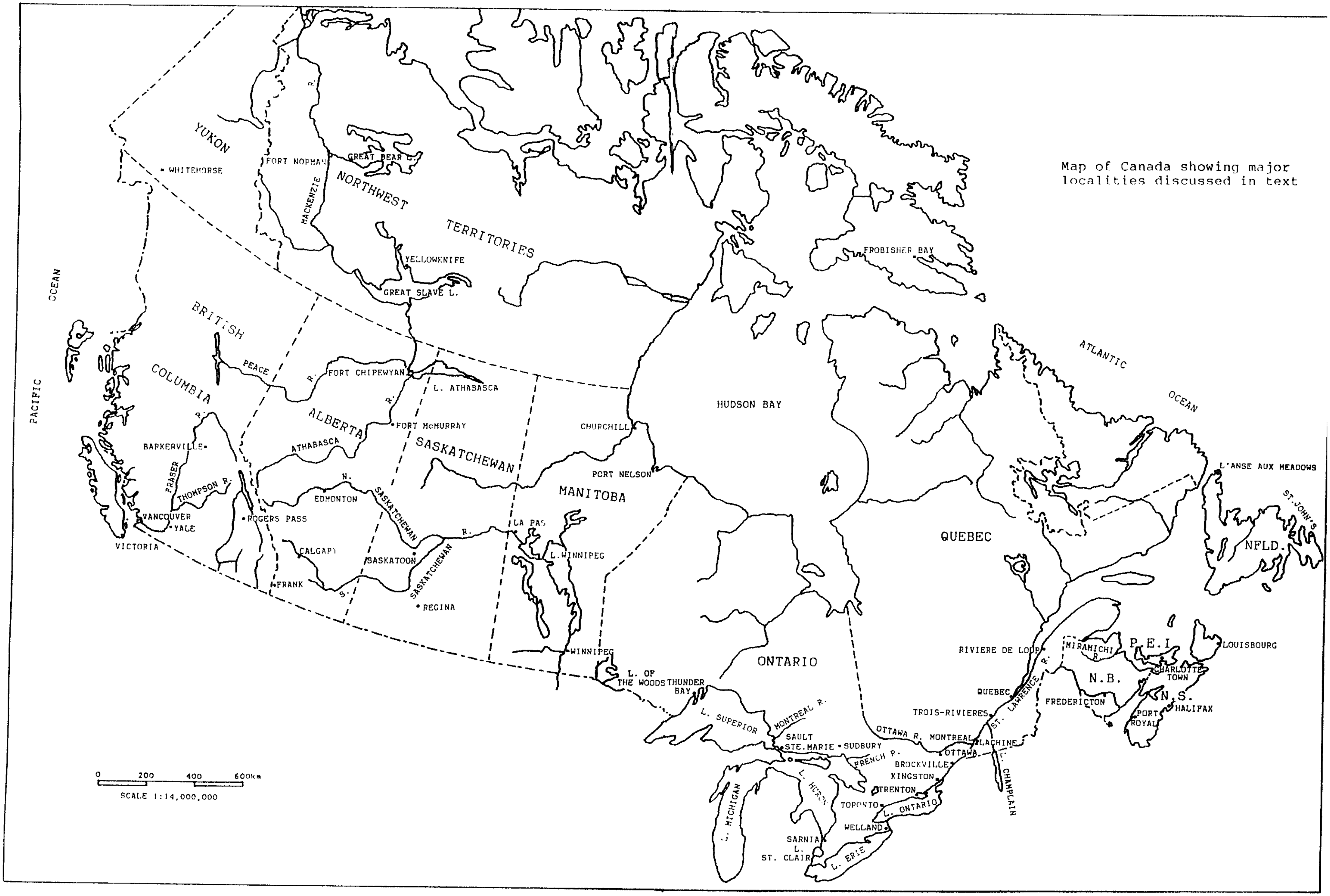

Map of Canada showing major localities discussed in text

Geotechnical Practice in Building Construction

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SUMMARY

This paper describes the current state of practice in principal applications of geotechnical engineering in Canada and in particular in building construction. The topics covered are geotechnical investigations, shallow foundations, deep foundations, temporary excavations and groundwater control.

RESUME

Le présent article expose l'état actuel de la pratique canadienne dans les principales compétences de la géotechnique dont fait appel la construction de bâtiments. Les sujets traités comprennent les reconnaissances géotechniques, les fondations superficielles et profondes, les fouilles temporaires et l'épuisement des fouilles.

INTRODUCTION

The principal role of the geotechnical engineer in building construction is largely related to the design and construction of foundations. Geotechnical engineers select adequate foundations for a proposed building and a suitable method for constructing its foundations. They must therefore find a safe and economical foundation design that fits the conditions of the site and the structural and long term behavioural requirements of the building. During construction, they assure the safety of the excavation and the safety of adjoining structures by selecting the right excavation method, and they ensure the short-and-long term performance of the foundations by specifying suitable construction and installation requirements.

In assessing the conditions of a given site, geotechnical engineers carry out various tests and collect data from other sources. They evaluate the data in the light of their prior experience and that recorded by others. Such experience is generally compatible with rational theory or, where no theory exists, with observation records of actual structures. In assessing the data, their judgment must always be precise.

This paper elaborates on some major aspects of geotechnical engineering practice in building construction, particularly in geotechnical investigations, foundation design, installation of foundations, temporary retaining structures, excavations and dewatering. More detailed information on Canadian practice and procedures in foundation engineering is found in the Canadian Foundation Engineering Manual (1985) which was used as a reference in preparing this paper.

GEOTECHNICAL INVESTIGATIONS

The geotechnical investigation is the first important step in a foundation design. Its ultimate goal is to obtain adequate and reliable information on the geotechnical conditions to select a safe and economical foundation for a given site and structural conditions and to assess its probable behaviour when loaded.

For the overall economy of the project, many of the geotechnical data needed for construction are developed during the geotechnical investigation for foundation design. Thoroughness is essential in a geotechnical investigation to achieve safety and maximum economy.

Objectives

In site investigation, subsurface conditions are appraised by analysing information gained by such methods as geological and geophysical surveys, borings and sampling, in situ testing, visual inspection, laboratory testing of samples of the subsurface materials, and groundwater measurements and testing.

This information supplements, whenever possible, previous analyses of existing data in the form of geological, geotechnical and topographical maps and reports and aerial or satellite photographs of the area. Local experience is especially valuable in many aspects of the study, including such environmental factors as frost depth, swelling conditions and the behaviour of any neighbouring existing foundations. For a complete foundation study, information is required on the sizes and overall purpose of the proposed structure, the loading conditions that may be applied to individual parts of the foundations and the type and period of construction.

With the help of the background data, the specific objective of a geotechnical investigation is to determine the nature and sequence of the soil strata at the site; the groundwater conditions; the physical properties of the soils and rock; the mechanical properties, such as the strength and deformation characteristics of the different soil or rock strata; and other specific information, when needed, such as the chemical composition of the groundwater, and the characteristics of the foundations of adjoining structures.

Scope of Investigations

Since one is dealing with the unknown, and given that the site under study may be extremely variable, no firm rules can be established for the spacing and number of borings and soundings required to arrive at the objectives mentioned previously. Where site conditions are relatively uniform or where local knowledge and background information is of high quality, the number of borings and soundings and the frequency of sampling can be reduced. The number and type of borings and soundings must leave no doubt as to the complexity or uniformity of the site.

Unless bedrock is encountered first, the site investigation must reach the depth where vertical stresses induced by the proposed construction is less than 10% of the existing overburden stresses at that level. The general guidelines to the depth of investigation are as follows:

> It is good practice to have at least one boring to bedrock, or to well below the anticipated level of influence of the proposed building. Bedrock is proved by coring into it to a minimum depth of 3 m.
>
> For light structures, insensitive to settlement, the borings should reach a depth equal to four times the probable footing width or to a depth of 6 m below the lowest part of the foundation, whichever is the deeper.
>
> For more heavily loaded structures, such as multi-storey structures and framed structures, at least 50% of the borings should be extended to a depth equal to 1.5 times the width of the building below the lowest part of the foundation.

The frequency of sampling and in situ testing varies with the material being tested and the purpose of the site investigation. The more variable the geotechnical conditions, the greater the number of samples and in situ tests. Generally, the frequency of sampling and testing ranges from continuous to once every metre a short distance below foundation level, then decreasing to intervals of 1.5 to 3.0 m toward the base of the borings. A sample is usually taken at the upper contact of each stratum.

Samples may frequently alternate with in situ tests in an almost continuous way, particularly in cohesive soils. The frequency of sampling and in situ testing is not always determined before the first few borings are completed, and is often developed as the field investigation program progresses. Where local knowledge and background information are well documented, the sampling and in situ testing frequency can generally be predetermined. It will vary for each site investigation.

Sampling Methods

The soil samples recovered in the process of a geotechnical investigation can be classified into four classes. Table I shows this classification based on the quality or the degree of disturbance expected with various types of sampler or sampling method in current use and gives the geotechnical properties which can generally be measured with any degree of reliability by laboratory testing on recovered samples. The choice of the type of sampler is partly related to the measurement of the soil properties. It is strongly influenced by the conditions of soil to be sampled and, to a lesser degree, by the cost of a particular type of sampling procedure.

Borings are the most common means of investigating soil conditions. Test pitting is a current method to obtain block samples in cohesive soils and also to examine and sample deposits that are difficult to sample otherwise.

Block samples are generally restricted to cohesive soils, although in some instances they are used for testing some of the

TABLE I

Classification of Soil Samples

Class	Quality	Identification	Properties that can be measured	Footnote No.
1	Undisturbed	a-Block samples	A,B,C,D,E,F,G,H,I,J,K	1, 2, and 4
		b-Stationary piston sampler	A,B,C,D,E,F,G,H,I,J,K	3
2	Slightly disturbed	Open thin-walled tube sampler	A,B,C,D,E,F,G,H,I	
3	Substantially disturbed	Open thick-walled tube sampler,such as a 'split spoon'	A,B,C,D,E,G	
4	Disturbed	Random samples collected by auger or in pits	A,C,D,E,G	5

A - Stratigraphy
B - Stratification
C - Organic content
D - Grain size distribution
E - Atterberg limits
F - Relative density
G - Water content
H - Unit weight
I - Permeability
J - Compressibility
K - Shear strength

NOTES

1 Block samples are best when dealing with sensitive, varved, or fissured clays. Wherever possible, block samples should be taken in such soils.

2 Samples of Class 1 are best stored in a vertical position in a room with constant humidity of minimum 80% and constant temperature of maximum 10°C.

3 Testing should occur as quickly as possible after sampling. Whenever possible, testing should be performed immediately after extrusion.

4 Because of inevitable stress relief, samples of all classes may be disturbed. The disturbance depends on the consistency of the sampled soil and increases with depth of sampling.

5 Water-content samples should be taken from freshly cut faces of a pit as the pit is advanced. Small diameter spiral augers are suitable for obtaining water-content samples of cohesive soils, if care is taken to remove free water from the sample, as well as all soil scraped from upper layers in the wall of the borehole. Water-content samples should be placed immediately in air-tight containers to prevent evaporation.

(from Canadian Foundation Engineering Manual, 1985)

softer rocks. They are usually limited to shallow depths and are labour intensive, implying a high overall cost. As a result, this type of sampling is not an everyday occurrence.

In soft to firm cohesive soils, the stationary piston sampler and the thin-walled tube sampler, also referred to as the Shelby tube sampler, are by far the most common types of samplers. There are many sizes available, most of them being in the 50 to 75 mm diameter range. The 75 mm piston sampler is the most commonly used sampler to obtain undisturbed samples of clays of a quality suitable for testing strength and deformation characteristics. Larger diameter samplers are, however, available and, for samples of sensitive clay, give laboratory test results of a quality close to those obtained on undisturbed block samples.

Samples of very stiff and hard clays are taken by means of triple tube samplers in which a thin-walled non-rotating inner tube is forced into the soil while the outer rotating barrel cuts the soil surrounding the sampler. A slightly disturbed sample is recovered inside a thin-walled sleeve for easy handling. This sampling technique is used whenever the resistance of the soil is too great for sampling with thin-walled tube samplers (ASCE, 1976).

In granular soils, the most common type of sampler is the split spoon sampler. The standard size split spoon has an outside diameter of 51 mm and an inside diameter of 37.5 mm. It is used to collect a disturbed sample and also to determine the standard penetration test N value.

Larger diameter split spoon samplers are available and occasionally used when the grain size of the material being sampled is too coarse for efficient recovery with the standard split spoon sampler.

Auger samples are generally completely disturbed, regardless of the size of the auger or the depth from which the sample is derived. Many augers are available, and may range from a small hand auger to motor propelled models, the most common sizes being in the 100 to 250 mm range. The auger sample is one of the most economical but the geotechnical information obtained from it is indeed limited.

In Situ Testing

The geotechnical properties of soils are determined either by in situ or laboratory testing or a combination of both. The determination of soil properties by means of in situ methods has developed rapidly in recent years (Robertson, 1986). Common in situ testing techniques are given in Table II, with their respective uses and limitations.

The standard penetration test (SPT) is the most widely used in situ test in geotechnical investigations. The test results provide a qualitative assessment of the properties of the soil, such as the compactness, and give a sample of the soil for description and classification purposes. The standardized test uses the 51 mm O.D. split spoon sampler which is driven into the soil at the base of an exploratory boring with a 63.5 kg weight having a free fall of 760 mm. The number of blows to drive the split spoon sampler a distance of 300 mm, after an initial penetration of 150 mm, is referred to as the N value. It is recognized that improper drilling and sampling procedures can greatly affect the N value of the standard penetration test.

The dynamic cone penetration test and the static cone penetration test are frequently used in site investigations. Both tests provide a qualitative logging of the soil stratigraphy. They also give a qualitative measure of some geotechnical parameters useful for foundation design. The most significant recent development in static cone penetration testing is the addition of a pore pressure gauge at the cone point. Pore pressure measurements provide more details on the stratification and add a new dimension to the interpretation of geotechnical properties.

The field vane shear test is the most common method for measuring the undrained shear strength of clays. The vane tester, best suited for soft to firm clays, measures the maximum torque necessary to rotate a small four-bladed vane attached to the end of a rod and pushed into the undisturbed clay. This torque is taken equal to the moment developed by the shear strength of the clay acting over the surface of a cylinder with radius and height equal to that of the vane. The obtained value is adjusted as a function of the plasticity of the clay to derive the undrained shear strength used for design.

The pressuremeter test is an in situ loading test carried out in a borehole by means of a cylindrical and expandable probe. The test allows the determination of the strength and load-deformation characteristics of the tested soil near the borehole wall. The parameters obtained are used to evaluate the bearing capacity. The pressuremeter is a useful tool for the investigation and design of foundations, particularly when dealing with soils that are difficult to investigate by more conventional geotechnical methods, such as dense granular soils, tills, soft rocks and frozen soils. It is also useful for determining design parameters for horizontally loaded foundations.

Groundwater Testing

Among the most important aspects of dealing with soils are the groundwater conditions. In fact, it has often been said that if there were no water, there would be no particular difficulties in dealing with soil and bedrock.

Groundwater may exist as the phreatic water table, a perched water table above an impermeable stratum, or a confined aquifer between two impervious layers, which is referred to as artesian water.

In carrying out borings, all water occurrences are noted, whether they be a water gain or loss. The following groundwater conditions are determined in current geotechnical investigations:

the exact water level(s) and whether the observed water levels are of the perched, phreatic or artesian type;

potential changes which may be expected to occur in the observed water levels;

the geological medium in which each aquifer is present and the order of magnitude of the water quantity that may have to be dealt with.

Provided drilling mud has not been used, a crude measurement of in situ permeability is obtained by pumping water into or out of a boring while it is in progress. Results obtained, however, are of limited dependability and quantitative values may be off

TABLE II

Common In Situ Tests

Type of test	Best suited to	Not applicable to	Properties that can be determined	Remarks
Standard penetration test (SPT)	Sand	Soft to firm clays	Qualitative evaluation of compactness. Qualitative comparison of subsoil stratification.	One of the most common tests, but must be used with good engineering judgment.
Dynamic cone test	Sand and gravel	Clay	Qualitative evaluation of compactness. Qualitative comparison of subsoil stratification.	As above, but not used on its own.
Static cone test	Sand silt, and clay		Continuous evaluation of density and strength of sands. Continuous evaluation of undrained shear strength in clays.	Test best suited for foundation design in in sand. Usage not widespread in Canada.
Field vane test	Clay		Undrained shear strength.	Most common in situ test for clay.
Pressuremeter test	Soft rock, sand, gravel, and till	Soft sensitive clays	Bearing capacity and compressibility.	Gradually gaining acceptance and conside-red appropriate for properties shown.
Plate bearing test and screw plate test	Sand and clay		Deformation modulus. Modulus of subgrade reaction. Bearing capacity.	Plate bearing test is relatively common but usually used where soils conditions are uniform and for shallow depths.
Flat plate dilatometer test	Sand and clay	Gravel	Empirical correlation for soil type, K_o, overconsolidation ratio, undrained shear strength, and modulus	Relatively new test.
Permeability test	Sand and gravel		Evaluation of coefficient of permeability.	Tests in borehole have limited accuracy. Results from pumping tests reliable to one order of magnitude.

(Adapted from the Canadian Foundation Engineering Manual, 1985).

by an order of magnitude or more. If required, more accurate measurement of the permeability of a water bearing stratum can be obtained by running a pumping test in a well. During the test, the drawdown of the water level is measured by piezometers placed at various distances from the well.

If more than one aquifer is suspected, piezometers are set in each aquifer and isolated from other aquifers by suitable impervious seals, since different piezometric levels may sometimes exist in the several aquifers. If deep excavations calling for dewatering are required, it is important to locate all aquifers and measure their piezometric levels to a depth of at least 1.5 times the depth of the lowest excavation below the highest groundwater level observed. If there is reason to suspect artesian pressures the investigations are extended even deeper.

Samples of groundwater are submitted to chemical analyses to determine whether deleterious materials are present. Of special concern is the presence of sulfates or sulfides which will attack concrete, or acids or salts which would accelerate corrosion of buried steel structures. If an extensive or long continued dewatering program is anticipated, tests of the groundwater will provide data on possible problems of encrustation of well screens or the occurrence of severe corrosion of certain types of well screens (ASCE, 1976).

SHALLOW FOUNDATIONS

A shallow foundation of a building generally derives its support from the soil or rock at a short distance below the building. The depth of the foundation below the adjacent ground surface is usually governed by the requirement to provide adequate protection against climatic or frost effects. Shallow foundations comprise such common foundation types as spread footings, strip footings and mats.

The geotechnical practice in shallow foundations includes the determination of the bearing pressures that may be safely applied to the foundation soil or rock, with due consideration to the settlement criteria of the proposed building and the natural environmental changes that may adversely effect the bearing properties of the soil or rock and the settlement of the building.

Foundation on Granular Soils

The design of a shallow foundation normally requires that both bearing capacity and settlement be checked. While either bearing capacity or settlement criteria may provide the limiting condition, it is normal for settlement to govern.

Bearing Capacity

The most current methods in Canadian practice for the determination of the bearing capacity of granular soils are noted and briefly described hereunder:

- Presumed allowable bearing pressure
- Shear strength parameters
- Standard penetration test
- Static cone penetrometer test
- Pressuremeter test

As many factors affect the bearing capacity of soils, there is no universally applicable value of allowable bearing pressure. Nevertheless, it is often useful to use a presumed allowable bearing pressure for preliminary design on the basis of the soil description. Such a value is reassessed for final design using a more precise method.

The bearing capacity of the soil may be calculated using the effective shear strength and unit weight parameters of the soil and the well known bearing capacity equation. For design, a factor of safety of 3 is conventionally applied to the calculated bearing capacity to arrive at the allowable bearing pressure.

A procedure frequently used to determine the bearing capacity of granular soils is the standard penetration test (SPT). The relationship between the SPT N value, the footing width and the allowable bearing pressure is available from different sources, the most current being those by Peck et al. (1974) and Meyerhof (1956, 1965). The allowable bearing pressures determined from these two sources are expected to produce maximum probable settlement not greater than 25 mm. It is generally recognized that the SPT results are greatly affected by the drilling and sampling operations making the SPT value less reliable. In engineering practice, the assessment of the SPT results and other related soil data requires good engineering judgment and experience for reliable foundation design.

In a known soil profile, the allowable bearing pressure may also be estimated from the results of a static cone penetrometer test. The allowable bearing pressure is evaluated from the cone point resistance by means of a simple empirical relationship.

The pressuremeter is used to determine the bearing capacity of foundations on soil or rock by means of in situ testing (Baguelin et al., 1978). The pressuremeter is particularly well suited for soils or rocks that otherwise are difficult to investigate, such as gravelly soils, glacial tills, and weathered or soft rocks. Foundation settlement can be estimated to an acceptable degree of accuracy by means of results from pressuremeter testing in granular soils or rock.

Settlement

The settlement of a building founded on a granular soil is normally estimated by empirical methods using the results of the above tests for bearing capacity determination. Alternatively, for more precise settlement predictions, a plate bearing test is carried out in which the settlement of a 300 mm test plate is measured and related to the expected settlement of a footing (Terzaghi and Peck, 1967). Settlement estimate is usually taken to mean the settlement directly related to the load, but this settlement generally occurs quite rapidly and frequently during the construction period. Post-construction settlement in such a case is small and considerably less than the predicted settlement.

However, post-construction settlement can occur even after a period of successful performance of the building as a result of changes in the groundwater level, like submergence or deep drawdown, or vibration from blasting, machine operations or other sources. Settlement of this nature, not included in the above estimate, must be assessed separately.

Foundations on Cohesive Soils

As a general rule, the construction of shallow foundations on cohesive soils requires care and experience because of the geotechnical properties of the soil and, more specifically, its compressibility and its sensitivity to remolding. For these reasons, elaborate sampling and testing of the soil are undertaken for site investigations involving clay. Complex analyses of the soil characteristics and the distribution of foundation stresses are needed to determine the allowable bearing pressures and the settlement predictions, and it is important that the construction techniques conform to good practice.

In current geotechnical practice, a shallow foundation must be designed to meet two requirements: it must be safe against shear failures of the supporting soil, and the settlement of the foundation must be less than the maximum tolerable settlement to preserve the integrity of the building and the functioning of sensitive operational features within the building.

Bearing Capacity

Current methods for the determination of the bearing capacity of clay include:

- Undrained shear strength test
- Vane shear test
- Pressuremeter test

For foundations in clay, the bearing capacity is governed by short-term stability conditions as given by the undrained shear strength of the clay. The bearing capacity is determined from a simple analytical relationship using the undrained shear strength of the clay within the zone of influence of the foundation loading approximately equal to twice the foundation width. To derive the allowable bearing pressure, a factor of safety is generally taken equal to 3 on the bearing capacity.

The relevant undrained shear strength is frequently measured by field vane shear testing. Alternatively, whenever the clay deposit is stratified with seams of granular soils or otherwise unsuitable for vane testing, the undrained shear strength of the clay is determined by laboratory testing on undisturbed samples. The current laboratory tests include the undrained triaxial compression test, the undrained direct shear test and the fall cone test.

The pressuremeter is also used to determine the bearing capacity for shallow foundations in clay as in other soil or rock conditions. It is, however, a testing method that requires a high quality borehole to provide the necessary uniform and undisturbed borehole wall and, for this reason, its use is generally restricted to soil conditions that are otherwise difficult to investigate (Baguelin et al.,1978).

Settlement

Settlement of a structure is the result of the deformation of the supporting soil. It is caused by elastic deformation or volume change owing to a reduction of the water content of the soil. Elastic deformation of the soil occurs quickly and is usually slight. It is normally ignored in the design of shallow foundations. Volume change associated with a reduction in the water content of the soil is called consolidation and can be estimated.

The consolidation characteristics of the clay used for calculating the predicted settlement of shallow foundations are obtained from laboratory one-dimensional consolidation tests on undisturbed and representative samples taken at various depths within the zone of foundation stresses. The test results provide the stress-strain relationship of the clay represented generally by the void ratio versus logarithm pressure curve as shown in Fig. 1.

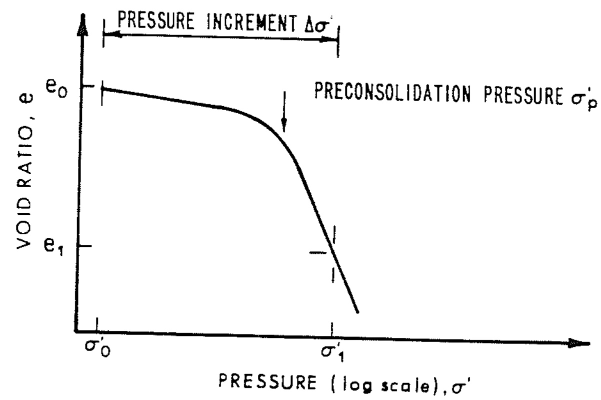

Fig. 1 Void ratio versus logarithm pressure curve

The settlement of a foundation on clay is calculated with less accuracy than the bearing capacity. Such a calculation is affected by a number of factors, the assessment of which requires engineering judgment. The most important of these is the preconsolidation pressure, that is, the maximum past pressure on the in situ soil.

If a foundation applies a pressure increment such that the final pressure is smaller than the preconsolidation pressure, the settlement of the clay layer will be small. If the pressure increment is such that the final pressure is greater than the preconsolidation pressure, much greater settlement will occur. The settlement in this case may be unacceptable for building foundations. Therefore, the preconsolidation pressure is a key parameter in settlement analysis of clay deposits and in the selection of the allowable bearing pressure for shallow foundations.

The settlement is determined by dividing the soil into layers under the foundation level, and computing the initial and final effective vertical stresses (or pressures) at the mid-point of each layer, whereupon the settlement for each layer is calculated using a well known analytical formula. The total settlement is the sum of the settlement calculated for the individual layers. All sustained loads added to the foundation soil such as backfill placed under or near the building are included in the calculation of the final effective vertical stresses.

Foundations on Rock

Rock is usually recognized as the best foundation material. However, there are dangers associated with unfavourable rock conditions, since overstressing a rock foundation may result in large settlement or sudden failure. A foundation on rock requires the same care as a foundation on soil.

In current practice, there are different methods for the determination of the allowable bearing pressure on rock for various ranges of rock mass quality as outlined in Table III.

TABLE III

Applicability of Methods for the Determination of Allowable Bearing Pressure on Rock

Basis of design method	Rock mass quality
Rock description	Sound rock and broken rock with wide or very wide spacing of discontinuities
Core strength	Rock mass with closed discontinuities at moderately close, wide, and very wide spacing
Pressuremeter	Rock of low to very low strength: rock mass with discontinuities at close or very close spacing
Soil mechanics approach	Rock of very low strength: rock mass with discontinuities at very close spacing

(from Canadian Foundation Engineering Manual, 1985)

For the first two methods, the following characteristics of the rock mass are determined from the geotechnical investigation:

- the identification and mapping of all discontinuities in the rock mass within the zone of influence of the foundation, including the determination of the aperture of discontinuities;
- an evaluation of the mechanical properties of these discontinuities, such as frictional resistance, compressibility, and strength of filling material; and the evaluation of the strength of the rock material.

For sound rock, the strength of the rock foundation is commonly in excess of the design requirements, provided the discontinuities are closed and are favourably oriented with respect to the applied foundation loads.

Allowable Settlement

All structures can tolerate some differential or total settlement without affecting their structural integrity or creating an aesthetic problem. Allowable differential displacement criteria in common use for buildings are given in Table IV.

TABLE IV

Allowable Displacement Criteria

Material	Maximum deflection between supports (L is the span length)
Masonry, glass or other frangible material	L/360
Metal cladding or similar nonfrangible finish	L/240
Steel or concrete frames	L/150 - L/180
Wood frames	L/100
Steel or concrete shear walls	by design
Structure	**Maximum slope of continuous structures**
High continuous brick walls	0.0050 - 0.0010
Brick dwellings	0.0030
Brick cladding between columns	0.0010
Reinforced concrete building frame	0.0025 - 0.0040
Reinforced concrete curtain wall	0.0030
Continuous steel frame	0.0020
Simply supported steel frame	0.0050

(from Canadian Foundation Engineering Manual, 1985).

Differential settlement occurs in all cases because of the natural variability of soils, even where total settlements are calculated to be uniform. The magnitude of these differential settlements may be related to the magnitude of the total settlements (D'Appolonia et al., 1968). Consequently, limiting the total settlement of a structure is an indirect means of controlling the amount of differential settlement.

Frost Protection of Foundations

For the design of shallow foundations, the foundations are placed beyond the depth of expected maximum frost penetration so that the soil or the rock beneath the bearing surface will not freeze. This measure alone, however, is not sufficient. Backfilling the excavation with soil that is frost susceptible may lead to damage resulting from adfreezing. The replacement of frost susceptible soil with granular material and proper drainage prevent adfreezing from occurring. Safe foundation depths on the perimeter of a building, therefore, are determined from the maximum depth of frost penetration. Interior foundations are generally placed at shallower depths in heated buildings.

The best indication of maximum depth of frost penetration in a particular locality is local experience. In the absence of local experience, the maximum depth of frost penetration can be estimated fairly closely by using one of the well known correlations between air freezing index and field observations of maximum frost penetration (Brown, 1964).

In recent years, lightweight plastic thermal insulation has been used to reduce the loss of ground heat and thereby reduce the depth of frost penetration. Insulation is of particular advantage in the design of unheated buildings such as warehouses, garages and refrigerated buildings used for food storage. It is also used to restrict the depth of frost penetration beneath artificial ice surfaces. Insulation can be obtained with relatively high compressive strength, so that it is possible to place slabs of these materials directly below the bearing surfaces of foundations (Robinsky and Bespflug, 1973).

Construction

The calculation of bearing capacity, the prediction of settlement and the choice of allowable bearing pressure may be grossly in error if the construction techniques do not conform to good practice. It is therefore part of current geotechnical practice to exercise quality control throughout construction of the foundations.

Inspection verifies that the foundation soil and the soil properties are the same as found in the geotechnical investigation and considered for the foundation design, and that the soil on which the foundations will be placed has not been remoulded by construction, inflow of water or other factors. In clays, and particularly in sensitive clays, it is important that all soils suspected to be disturbed by construction operations or otherwise be removed from the areas of foundations.

In winter construction, measures are taken to prevent frost from penetrating into the foundation soil.

DEEP FOUNDATIONS

A deep foundation provides support for a building by transferring loads either by toe-bearing to a soil or rock at considerable depth below the building, or by shaft friction, or both, in the soil or rock in which it is placed. Piles are the most common type of deep foundation. Piles are pre-manufactured or cast-in-place; they can be driven, jacked, jetted, screwed, bored, drilled or excavated. They are of wood, concrete, steel or a combination of all three. Some current features of piles are given in Table V.

The loads that may be applied safely to a deep foundation depend not only on the structural properties of the pile, but also on the properties of the foundation soil or rock and on the soil-pile system. One must distinguish the structural from the geotechnical capacity of a deep foundation. In many applications, geotechnical considerations may limit the permissible loads to levels well below those that might be arrived at on the basis of only structural considerations.

Geotechnical criteria for assessing permissible loads on a deep foundation are determined on the basis of site investigations and geotechnical analyses. The installation of piles may affect the geotechnical properties of the foundation soil. Also, the quality of a deep foundation may depend on the installation and construction technique, equipment, and workmanship. Such factors affecting the pile capacity are commonly verified and quantified by testing on actual foundation piles.

Different methods for calculating the geotechnical capacity of deep foundations are available for different geotechnical conditions and various pile installations.

Deep Foundations on Rock

Deep foundations placed on or socketed into rock normally carry heavy loads. They are driven or drilled and cast-in-place.

TABLE V

Some Features of Foundation Piles

Typical pile sizes, loads and uses	Material and structural considerations	Installation considerations
	WOOD PILES	
200 to 400 mm toe diameter. 6 to 15 m length; difficult to splice. Recommended as friction piles. Not recommended for hard driving through dense soil or to rock. Allowable loads, 100 to 500 kN.	Material and design must conform to subsections 4.2.3 and 4.2.7 of National Building Code (1985). Piles are treated when exposed to soil or air above permanent water level.	Low velocity hammer used (v.g. drop hammer). Hammer-rated energy limited to 160 000 J times the pile head diameter (in metres). Steel ring and steel shoe are used for hard driving.
	PRECAST AND PRESTRESSED CONCRETE PILES	
Diameter, 200 to 600 mm, prestressed and hallow cylindrical sections up to 1400 mm diameter. Up to 15 m (precast) and 40 m (prestressed) lengths without splicing; can be spliced. Suitable as friction piles, toe-bearing piles, uplift piles when designed for it. Allowable loads, 350 to 2500 kN.	Reinforced concrete must conform to subsection 4.2.3 of NBC (1985). Structural design must conform to subsections 4.2.4 and 4.2.7 of NBC (1985). Strength of splice is comparable to that of the pile. Splice is designed to maintain alignment of piles segments. Tolerance slack between two pile segments is less than 0.5 mm.	Drop and diesel hammers are commonly used. Possible tensile stresses in pile during soft driving and high compressive stresses during hard driving. Pile head is protected by a steel plate for hard driving. Steel shoe attached to pile toe for hard driving. Capblock and cushion are placed inside the driving helmet to prevent damage by direct impact.
	STEEL H PILES	
Sizes, 200 to 350 mm. Used for any depth; easy to splice. Optimal mill length: 12 to 21 m. Suitable for use as friction and/or toe-bearing piles. Allowable loads, 350 to 1800 kN.	Piles must conform to subsections 4.2.3 and 4.2.7 of NBC (1985). To minimize damage during driving, high yield strength steel is used. Increase steel section (1.5 mm) for corrosion allowance. Allowable stress usually not greater than 0.3 f_y.	Pile shoe usually used for hard driving. Various driving hammers are suitable. Rated energy of the hammer limited to 6 x 10^6 J times sectional area of pile (in m^2). Installation problems may be caused by too small sections.
	STEEL PIPE PILES	
Diameters up to 600 mm. Used for any depth; easy to splice. Driven with the lower end of the pipe open or closed. May be filled with concrete. Allowable loads, 350 to 1800 kN.	Steel pipe piles and concrete must conform to subsections 4.2.3 and 4.2.7 of NBC (1985). Allowable stress usually not greater than 0.3 f_y. Concrete contributes to pile capacity if driven to end bearing. Concrete has 120 mm slump or greater and is poured with an elephant trunk.	Rated energy of the hammer limited to 6x10^6 J times sectional area of the pile (in m^2). Wall thickness 5 mm, but 10 mm recommended to prevent buckling by lateral pressure and to achieve adequate capacity. For closed-end piles, use plate thickness of 10 to 20 mm. Driving shoe often used for hard driving.

TABLE V (cont'd)

Some Features of Foundation Piles

Typical pile sizes, loads and uses	Material and structural considerations	Installation considerations
	COMPACTED CONCRETE PILES	
300 to 600 mm shaft diameter. 3 to 18 m length. Suited in loose granular soils where high capacity can be developed at shallow depth. Unsuited in granular soils containing more than about 15 to 20% of fine grained soils. Allowable loads, 500 to 1600 kN.	Pile material and design must conform to subsections 4.2.3 and 4.2.7 of NBC (1985). Damp concrete (zero slump) is used in the compacted base. To resist uplift forces, a proper continuity of steel reinforcement is provided at the junction of the shaft with the base.	Installation requires special equipment and experienced contractor. Cannot be inspected. Contamination of concrete and reduction of shaft diameter (necking) may occur. Possible damage by adjacent piles. Encased shaft using a light-gauge steel tubing provides protection against intrusion of soil or water during concreting. Encased shaft is less subject to damage by adjacent piles than compacted shaft.
	BORED PILES AND CAISSONS	
Available in different shapes and sizes; cylindrical shape most common. Typical diameters, 300 to 3000 mm. Shaft sometimes permanently cased. Variable length are used. Best suited for toe-bearing high capacity piles to rock or very dense soil. Used in stiff clay. Allowable loads up to 18 000 kN.	Materials and design must conform to subsections 4.2.3 and 4.2.7 of NBC (1985). Concrete slump of 180 mm normally used. When bored piles excavated with bentonite slurry, the quality of slurry is controlled to insure good performance.	Excavation procedure depends on the soil and groundwater conditions. The base is cleaned to the sound founding material and uplift pressure is checked below founding level. Socket wall in rock is cleaned of loose rock and smear. Uncased shaft cannot be inspected except if temporary casing is used. Concrete pour must be fast and continuous. Placing of concrete is best done by pumping. Temporary casing is removed slowly to prevent intrusion of soil (necking) but before partial set of concrete.

A pile driven to or into bedrock cannot be inspected to determine the exact area of contact with rock, the depth of penetration into rock, or the quality of the rock at the foundation level. The load capacity of piles driven to rock is determined on the basis of observations during driving, load tests and local experience. As a first approximation, the capacity of such a pile is estimated on the basis of the unconfined compressive strength of the rock material (Rehnman and Broms, 1971).

Deep foundations can be drilled or excavated and cast-in-place. In this case, the rock conditions are verified and the allowable bearing pressure is determined on the basis of the foundation stratum in the same way as for shallow foundations. For sound rocks, the allowable bearing pressure is established on the basis of the unconfined compressive strength of the rock material (Ladanyi and Roy, 1971). For soft and stratified rocks, the pressuremeter is used to determine the bearing capacity of the rock (Menard, 1965; Baguelin et al., 1978).

Frequently, cast-in-place deep foundations are socketed into rock. Canadian practice for the design of such deep foundations varies from region to region and three different approaches are commonly used:

> The capacity is assumed to be derived from toe-bearing resistance only. However, if the bottom of the excavated socket is not properly cleaned, the capacity may not be mobilized without

large settlement.

The capacity is assumed to be derived from the adhesion between the concrete and rock along the surface perimeter of the socket. The capacity is highly dependent on the type and elastic properties of the rock and on the quality and roughness of the socket wall.

The capacity is assumed to be derived from both toe-bearing resistance and shaft resistance. This approach is used when verified by full scale load testing or local experience.

Socket shear accounts for large portion of the total capacity. The socket shear depends on the strength of the concrete and the rock and on the roughness of the socket wall. Grooves cut in the socket wall enlarge the roughness of the concrete-rock interface and were found to increase the socket shear in weak rocks (Horvath et al., 1983).

Settlement

The settlement of drilled foundations founded on sound rock is generally negligible. Settlement of such foundations is normally associated with the presence of open joints or compressible mud seams in the rock. Where such conditions are anticipated, detailed investigations and/or loading tests are carried out.

In situ plate loading tests can be used to assess the settlement behaviour of a rock mass under a deep foundation. However, the cost of such tests is high and only justified for large projects or in cases where the structure is very sensitive to settlement.

Settlement may also result from the presence of mud or debris left between the bottom of the concrete shaft and the rock surface. Careful inspection of the bottom of each excavation is necessary to eliminate this difficulty.

Piles in Granular Soils

Piles most commonly used in granular soils are driven piles or compacted concrete piles. Driven piles are generally precast concrete piles, wood piles, steel H-piles and steel pipe piles with the lower end opened or closed.

Piles that are driven into granular soils derive their capacity from both toe-bearing resistance and shaft friction. The relative contributions of toe-bearing resistance and shaft friction to the total capacity of the pile depend on the density of the soil and on the characteristics of the pile.

It is generally recommended to use displacement piles in granular soils in order to further compact the granular soil and thus obtain higher shaft resistance resulting in a greater pile capacity.

The roughness and the taper of a pile shaft are two important characteristics which affect the shaft resistance. Generally, wood piles mobilize more shaft friction than concrete or steel piles and steel produces the least friction. A taper of the pile wall of about 1% may result in a shaft resistance increase of approximately 50%.

Capacity

There are various methods currently used for estimating the geotechnical capacity of a pile in granular soils, namely:

- The standard penetration test (SPT)
- The theory of plasticity
- The static cone penetration test
- Load testing of piles
- Dynamic testing of piles

A method frequently used to estimate the capacity of a pile in granular soils is the standard penetration test (SPT). A simple relationship correlates the pile capacity to the SPT N value, takes into account the length and diameter of the pile and makes a distinction between displacement piles and low or non displacement piles (Meyerhof, 1976). Because of the uncertainties about the results of the standard penetration test, the method must be used with caution and requires engineering judgment and experience. For design, a factor of safety of 4 is currently applied to the calculated capacity to determine the allowable load.

The capacity of a pile can be determined from the friction angle of the soil by using the theory of plasticity (or bearing capacity theory). The analytical method requires some geotechnical parameters: the friction angle of the soil close to the pile, the soil-pile friction, and the ratio between the horizontal to the vertical effective soil stresses at the pile shaft. A factor of safety of at least 3 is applied to any theoritical computation.

The capacity of a pile in granular soils can also be computed from the results of the static cone penetrometer test using the values of the point resistance and the average unit side friction.

The design of piles with these analytical or empirical methods, or similar ones, is subject to some uncertainty. Indeed, the soil properties cannot be measured with great accuracy and are variable within a building site. The correlation between the soil parameters and the bearing capacity of a pile includes a margin of error. And, finally, the pile installation often affects the soil properties.

For these reasons, the most reliable

methods for determining pile capacity is by load testing piles. Whenever it is not practical to load test piles during the design stage, it is good practice to test piles at the beginning of the foundation construction. Alternatively, piles can be tested and their capacity determined by dynamic testing, a technique which is described in a subsequent paragraph.

In some granular soils, such as fine sands or cohesionless silts, the capacity of driven piles may decrease after driving. This effect known as relaxation, has also been observed in piles driven to shale bedrock (Samson and Authier, 1986). On the other hand, the capacity of piles driven in some saturated sands and silts may increase after driving. This effect is known as freeze. Restriking piles a few days after their installation is normally carried out to check such changes in pile capacity.

Compacted Concrete Piles

Compacted expanded base concrete piles in granular soils derive their capacity from the densification of the soil around the base due to the installation technique. The capacity of such piles depends on the construction method. The pile capacity used in design is generally supported by documented local experience and pile load tests. This type of pile is suitable in granular soil containing less than about 15 to 20% of fine grained soil.

Pile Group

In general, piles are used in groups. In granular soils, the capacity of a pile group is normally defined as being the sum of the capacity of individual piles, although it is possible that a pile at the interior of a group of piles may have a higher capacity than an individual pile.

The capacity of the pile group must be reduced if there is a lower resistance soil stratum within a short distance of the base of the piles.

Settlement

Many factors, not included in theoretical analysis, influence the actual settlement of piles. Therefore, estimates based only upon considerations of the elastic properties of the soil and pile material are generally inaccurate and of no practical value. Instead, estimates of settlement of piles are based upon empirical relationships and results of pile loading tests.

For a single pile, the settlement observed during a loading test may be considered as an approximation of the long-term behaviour of the pile, since time effects are usually negligible in granular soils. When load testing is not available, the pile settlement can be estimated from empirical formulae (Vesic, 1970, 1977).

The settlement of a pile group is evaluated empirically and the methods available are less reliable than those for single piles. Some of these methods relate the ratio of the settlement of the pile group to that of a single pile with the size of the pile group and the diameter of the piles (Vesic, 1970). Another current empirical method relates the settlement of the pile group to the standard penetration test N value of the soil (Meyerhof, 1976).

Piles in Cohesive Soils

Types of piles commonly used in cohesive soils are driven piles and bored piles.

The capacity of piles driven into cohesive soils is governed by the friction or adhesion between the pile and the soil and, to a much lesser extent for soft to firm clays than for granular soils, by the toe-bearing resistance. Similar to piles in a granular soil, the shaft resistance of the pile increases with the roughness and the taper of the pile wall.

The adhesion between the pile wall and the soil is not always equal to the undrained shear strength of the soil. The driving of piles in a cohesive soil causes some soil disturbance which varies with the soil properties, the pile type, the geometry of the pile group, the driving method and the sequence. This disturbance or remolding results in a temporary loss of the soil strength, some of which may end up as a permanent loss. In sensitive clays, substantial remolding of the clay may be observed. In such cases, it is preferable to use low displacement piles to reduce remolding.

As the clay adjacent to the pile consolidates after pile driving, the effect of soil remolding diminishes with time . In some cases, complete regain in shear strength to the original undisturbed value has not been attained even after a considerable period of time. Because of the slow rate of regain of strength. in certain cohesive soils, it is good practice to delay load testing of piles until several weeks after pile driving.

In stiff to very stiff cohesive soils, there is considerable evidence to indicate that, in driving, a gap is formed between the pile and the soil. This gap is not always fully closed with time, thus minimizing the adhesion to the pile relative to the high shear strength of the soil.

Capacity

Two different approaches are generally used to evaluate the capacity of a pile driven in a cohesive soil with a soft to stiff consistency. In such soils, piles generally derive their capacity almost entirely from shaft resistance. It is current practice to confirm the pile capacity by load testing for foundation design.

The first approach consists in evaluating the shaft resistance based on the undrained shear strength of the clay. The method uses a relationship of the shaft resistance as a function of the undrained shear strength of the soil and the pile type (Tomlinson, 1957).

The second method used to evaluate the shaft resistance of a pile driven into clay is based on the effective stresses and follows the same approach as for granular soils (Meyerhof, 1976). The method requires the following geotechnical parameters: angle of internal friction of the soil, the angle of friction between the pile wall and the soil, and the ratio between the horizontal to vertical effective soil stresses at the pile shaft.

Piles driven in very stiff clay (undrained shear strength greater than 100 kPa) mobililize their capacity from both toe-bearing and shaft resistances. However the shaft resistance on such a pile cannot be accurately predicted because little is known of the effect of driving on the shaft resistance and on the final effective contact area between the clay and the pile wall. As a first approximation, the pile capacity can be established on the basis of the effective stress analysis, as for softer clays. Alternatively, one can use an experimental relationship between the undrained shear strength of the very stiff clay and a coefficient of adhesion (Meyerhof, 1976). The actual pile capacity, however, must be confirmed by load testing.

Bored Piles

Large-diameter bored piles with or without enlarged belled bases, or under-reamed shafts, are successfully used in very stiff clays or cohesive tills.

The toe-bearing capacity is generally calculated on the basis of the undrained shear strength of the soil. The resistance along the shaft can be evaluated based on the in situ shear strength of the soil and an adhesion coefficient along the wall of the pile (Skempton, 1959).

The relative contribution of the shaft resistance and the toe-bearing resistance in an actual foundation is a function of the rigidity of the pile and the compressibility of the clay around the shaft and below the base of the pile. The allowable capacity of a bored pile is frequently based upon permissible pile movement determined by load testing.

Pile Group

When piles are driven in groups in soft to stiff clays, it is common practice to consider a group efficiency of 70% in the design. In very stiff clays, the group effect is neglected in the determination of load capacity of pile groups.

Settlements

Settlement of pile groups in homogeneous clay is estimated by methods normally used for shallow foundations. The most common method considers that the load carried by the pile group is transferred to the soil through an equivalent footing generally considered at one third of the pile length from the pile toe (Terzaghi and Peck, 1967). The load is assumed to spread into the soil at a slope of 2 (vertical) : 1 (horizontal) under the assumption that the equivalent footing is the top of the frustrum of a pyramid. The settlement for the equivalent footing is calculated using the method for shallow foundations.

Negative Skin Friction

If piles are installed in or through a clay deposit which is subject to consolidation, the resulting downward movement of the clay around the piles, as well as of any soil above the clay layer, induces a downdrag force on the piles through negative skin friction. The magnitude of soil settlement needed to cause the negative skin friction is very small.

Depending on the length of the piles above and within the stratum of consolidating soil, the magnitude of the downdrag force caused by negative skin friction can be large and must be taken into consideration in pile design. The negative skin friction is a function of the effective stress acting on the pile and is computed in the same way as for the shaft resistance.

Piles subjected to drag load are designed by an analytical method which estimates the requirements for the structural capacity, the settlement and the geotechnical capacity of the pile (Canadian Foundation Engineering Manual, 1985; Fellenius, 1984). Two separate loading conditions are considered in the design: for structural capacity, the permanent and the drag loads without the live load; and for geotechnical capacity, the permanent and live loads without the drag load.

Structural Requirements

In most cases, the allowable load on a deep foundation is governed by geotechnical considerations. The allowable capacity of a deep foundation determined from structural considerations represents the maximum axial load which theoretically could be carried by the foundation. However, for a number of reasons, this load is generally less than could be applied to a comparable unit if used in the superstructure of a building. The actual placing of deep foundations often deviates from the position and alignment assumed in design. Once in place, deep foundations often can neither be inspected nor repaired and the placement of concrete in cast-in-place deep foundations frequently cannot be done with the same degree of control as in structural columns.

Driving Requirements

For driven piles, a suitable pile driving system is selected so that sufficient energy will be transferred to the piles to develop the expected pile capacity without exceeding the maximum permissible driving stresses in the piles. The maximum permissible driving stresses correspond to values slightly less than the yield strength for steel piles, the 30 day compressive strength for concrete piles and the crushing strength for wood piles.

A useful method for assessing the pile stresses during driving is the theory of wave propagation, also called the wave equation analysis. Wave equation analysis takes into account the characteristics of the hammer, the driving cap, the pile and the soil. It is used to select hammers, capblocks and cushions, to predict driving stresses and pile capacities, and to choose driving criteria.

It must be emphasized, however, that these results are, at best, only approximations. Wave equation analysis, like empirical dynamic pile formulae, calls for the exercice of judgment and experience. No such method is employed to provide definitive values concerning driving criteria or load characteristics of driven piles. In current practice, the wave equation is used to provide a range of results established with due consideration to the possible variation of the hammer-pile-soil system. It is good practice to calibrate the wave equation results by pile load testing or dynamic pile testing which measure the driving energy and stresses transferred to the pile and the soil response to the impact on the pile. The dynamic pile testing is described in a subsequent paragraph.

Piles Subjected to Horizontal Loads

The horizontal capacity of piles depends on the combination of the stiffness of the piles and of the resistance of the surrounding soil to horizontal loads.

For vertical piles subjected to horizontal loads, an approximate and usually conservative approach assumes that each pile can sustain an allowable horizontal load equal to the passive earth pressure acting on an equivalent wall with a depth and a width equal respectively to 6 and 3 times the pile diameter. Similarly, the lateral resistance of a pile group may be approximated by the soil resistance on the group calculated as the passive earth pressure over an equivalent wall. The lateral resistance of the pile group must not exceed the sum of the lateral resistance of the individual piles in the group. The passive earth pressure acting against the pile caps, the basement walls, are added to the lateral pile resistance.

More elaborate methods for determining the horizontal capacity of vertical piles are based on the horizontal strength and stress-strain properties of the soil which are generally determined by full scale tests on piles or by pressuremeter testing. This analytical approach considers the bending stresses in the pile, the pile head deflection and the soil capacity in horizontal loading (Broms, 1964a,b).

Where large horizontal loads must be resisted, or where very soft soils occur, it is common practice to install inclined piles. It is considered that the horizontal loads are resisted by the horizontal components of the axial capacity of the inclined piles.

Piles Subjected to Uplift Forces

For certain types of structure, the foundation piles must resist uplift forces. Such piles must have the necessary geotechnical resistance to pullout and the structural strength to carry the applied tensile stresses.

The uplift capacity of a pile is equal to the negative skin friction which can be mobilized along the pile shaft, in addition to the weight of the pile. Wrongly, one often assumes that the resistance mobilized along the pile shaft in uplift is equal to that mobilized in compression. Pullout tests have however shown that the shaft resistance in uplift is much lower than that in compression (Bozozuk et al., 1978; Fellenius and Samson, 1976; Fellenius, 1984).

To evaluate pile uplift capacity, a reduction factor of about 0.5 is applied to the shaft resistance of the pile in compression. It is good engineering practice to confirm the uplift capacity of piles by load testing.

The uplift resistance of a pile group is the lesser of the two following values: the sum of the uplift resistance of the piles in the group; or the sum of the shear resistance mobilized on the surface perimeter of the group plus the effective weight of soil and piles enclosed in this perimeter.

Load Testing

Load testing of piles is the surest method for determining the pile capacity. Depending upon the type and the size of the foundations, such tests are performed at different stages during design and construction.

The best method of designing pile foundations is to perform pile driving and loading tests. To be representative, the testing is carried out on the same type of pile, using the same installation procedure and equipment as for the actual construction.

During construction, it is good practice to perform load tests on representative piles at an early stage. The purpose of such tests is to ascertain that the allowable capacities obtained by design are appropriate, and that the pile installation procedure is satisfactory.

Routine load tests for quality control are currently carried out on representive piles to assess and verify the uniform safety of the allowable capacities and the behaviour of the constructed foundations. Load tests are performed on one of every 250 piles, or portion thereof, of the same type and capacity of pile, and on one of each group of piles where soil conditions differ.

The pile load tests are carried out according to standard methods for the arrangement, execution and reporting (ASTM D-1143, load test; ASTM D-3689, pull test; ASTM D-3966, lateral test).

There is a wide variety of criteria for interpreting the results of the pile load tests (Fellenius, 1975, 1980). The most appropriate criterion is selected on the basis of the type of pile tested and on the performance requirements of the foundations.

A factor of safety is applied on the pile capacity determined by loading tests to derive the allowable pile load. It generally varies between 2.0 and 2.5 depending on the representativeness of the test results.

Dynamic Pile Testing

More recently, dynamic testing of piles has been used in Canada to determine pile capacity and other engineering characteristics during pile driving. The system used is a pre-programmed field computer linked to transducers attached to the pile and measuring strain and acceleration during driving. The instrument converts the measured signals to force and velocity and solves for soil resistance, driving force and energy transmitted to the pile, maximum compression and tension stresses in the pile, pile integrity and others. (Goble et al., 1970; Rausche et al., 1972; Authier and Fellenius, 1983). With dynamic testing, toe-bearing resistance and shaft resistance of the pile can be evaluated separately.

One of the advantages of dynamic testing is that several piles can be tested in a day, and for the cost of one static loading test, to account for the natural variation of capacity between piles.

Inspection

The quality of a deep foundation is governed by the installation methods. A correct choice of installation procedure and equipment, good workmanship, and tight quality control of all installation work is essential to the construction of adequate deep foundations. Consequently, inspection is of utmost importance. It is good engineering practice to carry out continuous inspection during construction of deep foundations. Several features must be inspected to ensure good quality:

- the location and alignment of all foundation units;
- the integrity of piles and of pile components and materials and their conformance to specifications;
- the pile driving operations including the driving records, pile hammer used, pile set;
- the pile capacity by means of static load testing or dynamic testing;
- horizontal and vertical movements of adjacent piles during pile driving;
- vertical movements of adjacent shallow foundations during pile driving.

TEMPORARY EXCAVATIONS

The construction of foundations requires temporary excavations, and a geotechnical approach is used to determine their stability. The design of excavations, and particularly excavation faces, depends on several factors, the most important being the geotechnical and groundwater conditions, the depth of the cut and the distance to adjoining properties or existing structures.

Unsupported Excavations

It is usually more economical to keep the foundation excavations unsupported when their depths are limited and the construction site is large enough to provide stable side slopes.

Slopes of unsupported excavations may fail, with serious consequences. It is important therefore that the safety of an unsupported excavation be ensured by thorough geotechnical investigation and good design.

Excavation in Granular Soils

Instability of excavation slopes in granular soils is most frequently caused by water action, either by ground or surface water.

If the natural groundwater table is shallower than the proposed depth of excavation, the current practice for stable side slopes consists in lowering the groundwater table below the excavation bottom before and during the excavation by an appropriate drainage method. The lowering of the groundwater must be deep enough below the bottom of the excavation to prevent failure of the side slopes and instability of the excavation base.

Where the groundwater level is kept below the bottom of the excavation, the sides of the excavation are inclined at an angle smaller than the friction angle of the soil for stable slopes.

Water run-off can cause severe slope erosion in granular soils. This is usually prevented by diverting surface water from the excavated slopes. In high slopes, it is good practice to provide benches about every 3 metres, with drainage to catch and lead off run-off water. Recently, such products as geotextiles and geogrids placed on the slopes have proved successful in preventing erosion by rain or run-off water.

Excavation in Cohesive Soils

Excavation in cohesive soils poses more complex problems than in granular soils, and the slope stability must be more thoroughly analysed. In clay, deep rotational failure is possible affecting not only the sides but also the base of the excavation. Slope failures in clay frequently occur a few days or weeks after the end of excavation, at times when construction activities may well take place at the bottom of the excavation. In sensitive clays, retrogressive slides may develop following initial local failure, resulting in damage not only to the construction site, but to neighbouring properties.

A cut slope in clay might be stable for an initial period and then fail. Delayed failures in cut slopes are attributed to the progressive reduction of the shear strength of the clay caused by the slow equalization of the pore water pressure following the generation of negative pore water pressure by stress release during the excavation. The magnitude of reduction of the clay shear strength depends on the soil properties, particularly the degree of overconsolidation, and the rate of strength reduction rests on the stiffness and the permeability of the clay (Tavenas, 1984).

The important factors affecting the stability of a cut slope in cohesive soils are not only the depth of the cut, the soil and groundwater conditions but also, in some instances, the length of time the excavation stays open. For these reasons, the stability of a temporary cut slope in clay is analysed for both short-term and long-term conditions.

Short-term Stability

The short-term stability of cut slopes in clay is determined by total stress analysis using the undrained shear strength of the soil. The undrained shear strength of the clay is obtained from laboratory undrained triaxial compression tests on undisturbed samples or, whenever possible, from field vane shear tests.

The stability of cut slopes is analysed by well known numerical methods. For cases where the undrained strength of the soil does not vary much over the height of the slope, stability charts provide a simple means of determining a stable slope (Janbu, 1954).

Clay deposits usually present a stiff fissured crust near the ground surface. Upon excavation, the upper part of the slope is subjected to horizontal tensile stresses and the fissures tend to open. For the stability analysis, no strength is assumed in the fissured zone and full hydrostatic pressure is considered to act in the fissures unless adequate drainage or protection against water infiltration is provided.

Long-term Stability

The long-term stability analysis is carried out in terms of effective stresses using the drained shear strength parameters of the clay: the effective angle of internal friction, ϕ', and the effective cohesion intercept, c'. Groundwater within the slope and piezometric pressures at or below the toe of the slope may greatly reduce the resisting shear strength along a potential failure surface. The groundwater conditions must be investigated and the data used in the stability analysis.

The effective strength parameters of the soil are determined by laboratory triaxial compression tests or drained direct shear tests. The piezometric pressures are measured in the field through piezometers installed on the site at proper locations and depths. The stability analyses are generally carried out using the Bishop simplified method for circular failure surfaces (Bishop, 1955). Generally the factor of safety of cut slopes is 1.5 minimum.

It is current practice to design a temporary cut slope in clay considering both short-term and long-term stabilities.

In general, excavations will be more stable in short-term than in long-term conditions. The length of time required before the long-term condition becomes relevant to stability depends on many factors and cannot be predicted with the required accuracy for most practical cases. For this reason, both short-term and long-term stabilities are checked for the adoption of a safe excavation design.

It is good practice that major slopes be instrumented to monitor pore water pressures and slope movements. The instrumentation includes piezometers, survey points and slope inclinometer tubes. Before failure, significant deformation occurs in all slopes and this may serve as an indicator of incipient failure. Variations of the pore water pressure can be important and unfavourable to the stability of a slope.

Excavation in Rock

Because of their high strength, rock masses are usually suitable for unsupported vertical excavation faces. For excavations to the depths generally encountered in building construction, the stability of rock faces is usually evaluated by a survey of the pattern of bedding planes, joints and other discontinuities of the rock mass.

Where rock discontinuities are unfavourably oriented, local support is provided by rock bolts or rock anchors. Where discontinuities are spaced so closely as to permit the loosening and fall of small blocks, the work area is protected, for instance, by steel wire mesh attached to the face of the slope or by guniting. Where rock is highly weathered or fractured, it is treated as soil for the design of the excavation faces.

Supported Excavations

When sloping sides of a temporary excavation are unfeasible for lack of space, reasons of cost or other considerations, temporary supports are commonly used to hold the excavation faces.

The functions of an earth retaining structure in a temporary excavation are multiple. Its prime purpose is to retain the soil at the back of the retaining wall, including all surface surcharges that may be applied during the life of the structure. Frequently, an excavation wall may hold a water head which often exerts appreciable pressures on the wall. In the presence of nearby structures, the role of the retaining wall is at the same time to limit the settlement of the soil behind the wall to an acceptable magnitude. Finally, in certain geotechnical conditions, the earth supporting structure may have to be designed to ensure the stability of the base of the excavation.

To meet these requirements, the design and construction of an earth supporting structure call for several geotechnical considerations and some of these are addressed hereunder.

Design Earth Pressure

The different types of earth retaining structures can be classified into two broad categories for earth pressure considerations: the rigid walls such as basement walls, and the flexible and semi-flexible shoring walls such as anchored walls and strutted walls, commonly used for temporary supports.

For rigid walls, the earth pressure can be computed from theory and the pressure distribution is close to triangular. The magnitude of the earth pressure acting on the wall depends strongly on the lateral movement of the soil behind the wall. It ranges from the minimum value of the active case (represented by the active earth pressure coefficient, K_a) when the soil movement reaches the failure limit due to lack of support, to the at-rest case corresponding to no lateral soil displacement (coefficient of earth pressure at rest, K_o), and to the maximum value of the passive case when the soil movement reaches failure due to lateral thrust against the soil (passive earth pressure coefficient, K_p). The passive pressure is several times greater than the active pressure, as shown in Fig. 2. This figure also illustrates the important role of the lateral movement of the soil in the magnitude of the earth pressure.

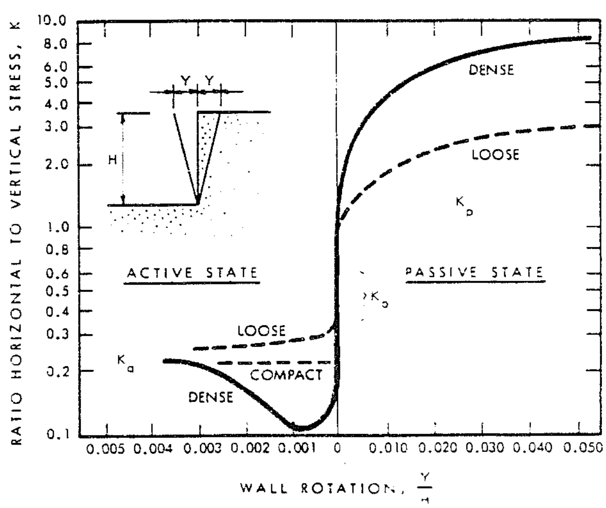

Fig. 2 Effect of wall movement on earth pressure in sand (from Canadian Foundation Engineering Manual, 1985).

For flexible and semi-flexible walls, the soil and wall deformations and, hence, the earth pressures are much more complex. The yield of one part of a flexible wall throws pressure onto the more rigid parts. Therefore, the pressures in the vicinity of supports are higher than in unsupported areas, and the loads on individual supports vary, depending largely on the stiffness characteristics of the supports and also on the construction procedure. In such cases, no satisfactory general theoretical solution for earth pressures is available. The design earth pressure is based on previous field observations and takes into account the method and sequence of construction and the tolerable movement of the sides of the excavation.

The lateral earth pressures normally anticipated are represented by pressure envelopes having a triangular, trapezoidal or rectangular form and vary according to the type of retaining wall and the nature of the soil. Fig. 3 illustrates the pressure distributions used in the design of strutted walls applicable to three soil types. It is the prevailing practice to consider the same pressure distributions for the design of anchored walls, including concrete diaphragm walls. The applicable lateral earth pressure coefficient ranges between the K_a active pressure and K_o at-rest pressure coefficients, depending on permissible wall and soil movements as discussed in a later paragraph.

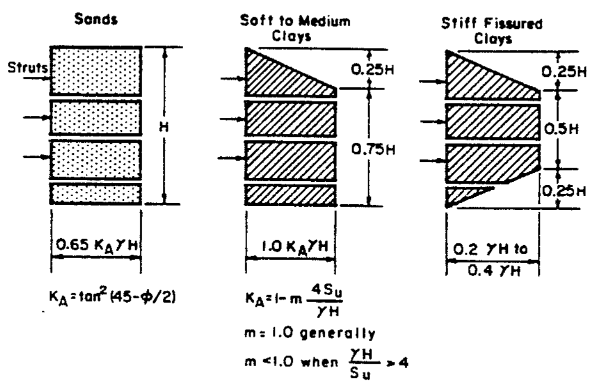

Fig. 3 Apparent earth-pressure envelopes for design of struts (from Terzaghi and Peck, 1967)

Any surface surcharge must be considered in the design of the wall and the resulting additional wall pressure can be determined by analytical procedures.

The excavation wall elements may be either open, permitting full drainage, or closed, providing an impermeable barrier. Closed systems are designed for soil and full groundwater pressures. The total pressure acting on the wall is thus equal to the water pressure added to the earth pressure computed using the submerged weight of the soil below groundwater level and the total weight above the water level. The hydrostatic pressure is not included in open systems where seepage through the wall can take place.

Cantilevered walls are sometimes used for temporary support of soil faces of relatively low height. Because of their flexibility, the earth pressures acting on cantilevered walls are considered to approximate the active condition with a triangular distribution.

Movements Associated with Excavation

Movements associated with excavations are primarily related to construction technique and commonly consist of lateral yield of the soil and support system towards the excavation with corresponding vertical movement of the adjacent ground surface. Both lateral and vertical movements due to yield are normally of the same order of magnitude. Where the quality of construction is poor, erratic movements can occur due to loss of ground or erosion behind the wall.

For well constructed support systems, it has been found that movements are dependent on the wall height and are related to the soil type. Table VI summarizes the approximate range of vertical and lateral movements to be expected. It should be recognized that, in certain cases, more favourable results may be achieved with good design, good construction workmanship and careful field supervision, including the monitoring of behaviour.

In strutted wall excavations, movements are controlled by construction details and procedures. Such movements develop in each excavation phase before the next level of struts is installed. Movements can be reduced most effectively by limiting the vertical spacing between struts and by prohibiting excavation below strut level until the struts are installed, properly blocked and prestressed.

The yield movements of anchored walls are controlled more by design methods than with strutted walls. The number of anchors and the vertical spacing of such anchors play a significant part in controlling the degree of lateral deformation. When greater control of adjacent ground movements is required, the earth pressure is computed using the at-rest K_o earth pressure coefficient and the anchors are prestressed to full design loads.

In normal practice, movements due to yield of anchored walls are usually less than for strutted walls for the same depth of excavation. Limited experience suggests that concrete diaphragm walls induce smaller movements than sheet pile or soldier pile walls.

Basal stability

Soft to Firm Clays

Deep supported excavations in soft to firm

TABLE VI

Vertical and Lateral Movements
Associated with Excavation

Restraint	Wall details	Granular soils, % depth	Stiff clay, % depth	Soft to firm clay, % depth	Remarks
Canti-lever	Conventional stiffness	Moderate to large	Moderate	May collapse	Movements related to wall, soil stiffness and embedment condition.
Braced	Soldier piles or sheet piles	0.2 to 0.5	0.1 to 0.6	1 to 2	Struts installed as soon as support level reached and prestressed to 100 per cent design load.
	Rakers or struts loosely wedged	0.5 to 1.0	0.3 to 0.8	>2	Poor workmanship would result in greater values.
Tied-back	Soldier piles or sheet piles	0.2 to 0.4	0.1 to 0.5	1 to 2	Prestressed to pressure between active and at-rest.
	Concrete diaphragm walls	<0.2	<0.1 to 0.5	<1 to 2	Presstressed as above, since wall stiffness and design earth pressures are normally greater, movements generally are less than for soldier piles or sheet piling; little data available.

NOTES

(1) Indicated movements apply directly behind wall; for granular soils and stiff clays, movements would be expected to feather out in approximately linear fashion over horizontal distance of 1.0 to 1.5 depth of excavation (H). For soft to firm clays, and assuming average workmanship, this distance increases to 2.0 to 2.5H, and with poor workmanship to greater than 3H.

(2) If groundwater is not properly controlled in granular strata, movements may be much larger than indicated, and loss of ground could also result.

(3) If the factor of safety against base heave for soft to firm clays is low, large deformations will result.

(4) Upper range of movements usually applicable for highly sensitive clays in either stiff or soft to firm category.

(5) Experience indicates that movements are reduced by using close vertical spacing between strut or tie-back levels and by careful attention to prestress details.

(from Associate Committee on the National Building Code, 1985).

clays may fail by base heave as a result of excessive shear stresses in the soil near the base of the excavation. Base heave failure is associated with loss of ground and appreciable surface settlement adjacent to the excavation.

The basal stability is determined using a simple analytical method and the undrained shear strength of the clay at the base of the excavation. To prevent substantial deformation of the soil adjacent to the excavation wall in soft to firm clays, the factor of safety against base failure must be greater than 2 (Janbu, 1954).

The effectiveness of the sheeting in reducing deformation of the excavation base is increased appreciably by driving the sheeting deeper below the bottom of the excavation, into a hard stratum whenever possible.

Granular Soils

In granular soils, basal instability takes the form of piping or heave and is associated with groundwater flow under upward gradient.

Groundwater control is achieved by drainage or by lowering the temporary retaining wall below the base of the excavation to a sufficient depth to provide an adequate cut-off, or by a combination of both methods.

CONTROL OF GROUNDWATER IN EXCAVATIONS

The objective of groundwater dewatering in temporary excavations is to lower the water table below the bottom of the excavation or to reduce the water pressures in underlying pervious strata to ensure the stability of the excavation.

Dewatering Methods

Three basic methods are currently used for dealing with groundwater in construction excavations:

The groundwater is allowed to flow into the excavation where it is pumped out from sumps and ditches.

The water is lowered in advance of excavation using wells, wellpoints or other pre-drainage systems.

The groundwater flow is cut off at the periphery of the excavation with sheet piling, diaphragm walls, slurry trenches, tremie seals or grout.

To select the dewatering method to be used for any specific case, adequate information is required on the many factors affecting the suitability of a given method. These include:

soil conditions, geological stratigraphy, permeability of the pervious strata;
groundwater conditions, namely the depth of the groundwater table and the piezometric levels in underlying pervious strata;
the depth and size of the excavation;
the method of supporting the sides of the excavation, i.e. open or supported excavations;
the proximity of existing structures, their type and depth of foundations, and their sensitivity to groundwater lowering.

It is important to recognize that flexibility in the dewatering plan is vital to its success. No matter how thorough the geotechnical investigation, some unforeseen condition may develop and cause serious difficulties, if not identified early and adjusted for.

Open Pumping

Open pumping from sumps and ditches is usually the least expensive method. Under favourable conditions, it is a satisfactory procedure. These favourable conditions are mostly related to the soil characteristics (e.g. dense, well-graded, or cemented soils), the hydrogeological conditions, the excavation methods and the excavation supports such as flat slopes in open excavation.

Pre-drainage

When it is necessary or advisable to lower the water level in advance of excavation, there are several techniques for the purpose.

Wellpoints are most suitable in shallow aquifers where the water level needs to be lowered no more than 5 to 6 metres. Beyond that depth, multiple stage wellpoints are required.

Deep-well systems are also suitable for lowering the groundwater table. They are most suitable where deep lowering of the water level is required and where the soil formation becomes more pervious with depth.

Cut-off Methods

The methods available for water cut-off vary greatly in their effectiveness and in their suitability for installation in various soils as shown in Table VII.

If the sheeting or diaphragm does not penetrate into an impermeable layer, flow will occur under the sheeting or diaphragm and up into the excavation. Unless groundwater control is adequate, this flow will cause instability of the excavation base, generally referred to as piping, heave, or quick conditions.

In some cases, appropriate analysis of the seepage flow may indicate that pumping out of the seepage water may be facilitated and stability improved by covering the bottom of the excavation with a properly graded sand and gravel filter blanket.

TABLE VII

Suitability of Cut-off Methods

Soils conditions	Slurry trench	Dia-phragm walls	Sheet piling	Grou-ting
Silty sand and clayey sand	Yes	Yes	Yes	Poor
Coarse sand and gravel	Yes	Yes	Vari-able	Yes
Stratified soil	Yes	Yes	Yes	Poor
Boulders	Fair (can be costly)	Costly	Poor	Yes
Rock	Costly	Costly	No	Yes

Methods of Pressure Relief

Where an excavation is dug into an impervious deposit underlain at relatively shallow depth by a pervious stratum, the piezometric head in this stratum and the seepage may lead to basal instability of the excavation. In such a case, it is necessary to lower the water head in the deep pervious stratum to a safe level by means of relief wells. Fig. 4 illustrates this case, which may occur in open or supported excavations and in various geotechnical situations including zones of permeable rock.

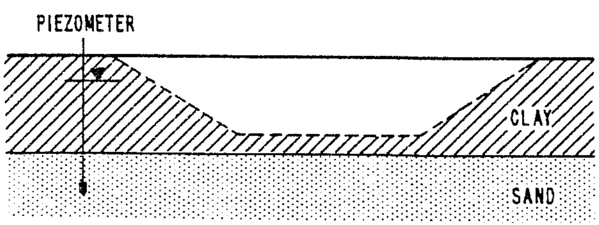

Fig. 4 High piezometric pressure below excavation

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Geotechnical Practice in Mining

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SUMMARY

There is a long-standing relationship between the geotechnical profession and the mining industry that has developed over many years of service by geotechnical engineers to assist in the economic extraction of mineral resources. This paper describes the broad nature of this association.

Some of the services provided are project related, involving the construction of surface facilities required for the general operation of the mine. Others form a direct part of mine planning and production activities for open pit and underground mines. In all cases, the relationship has reached a significant degree of maturity where high quality geotechnical expertise is relied upon by the mining industry. The needs of the mining industry provide an important challenge to the geotechnical profession to assist the Canadian mining industry to remain competitive and adopt innovative solutions to novel problems.

RÉSUMÉ

Entre la profession géotechnique et l'industrie minière, il y a des relations très anciennes, développées au cours des nombreuses années où les ingénieurs ont travaillé à la mise en valeur des ressources minérales. Cet article décrit la nature générale de cette association.

Certains des services fournis sont reliés à des projets impliquant la construction des installations de surface requises pour l'opération de la mine. D'autres font partie directement des activités de planification et de production des mines souterraines ou à ciel ouvert. Dans tous les cas, les relations ont atteint un degré élevé de maturité, l'industrie minière comptant sur une expertise géotechnique de haute qualité. Les besoins de l'industrie minière constituent un défi important pour la profession géotechnique, qui doit aider cette industrie canadienne à rester compétitive, et à choisir des solutions originales aux nouveaux problèmes.

INTRODUCTION

The geotechnical and mining professions have maintained a long history of close association. While in some cases this association is slightly removed from the direct mainstream excavation of raw materials, geotechnical engineering in all its facets has proven to be vital to achieve the high standard of safe, economic, innovative and socially responsible mining for which the industry is noted.

This paper highlights the inter-dependency of the mining industry and the geotechnical profession, and exemplifies the broad spectrum of mining activities which rely on geotechnical expertise. The experiences and biases that have developed through the author's own association with the mining industry naturally colour the discussion, reflecting a dominant theme of rock mechanics and ground control for underground mining. The inter-relationship between geotechnical and mining engineering is very much broader than this however, and an attempt is also made to highlight the multi-facetted nature of the association.

WHERE THE MINING AND GEOTECHNICAL PROFESSIONS MEET

Most mining developments are comprised of the following three components:

- Surface infrastructure;
- Surface facilities subsidiary to mainstream production;
- Direct production aspects.

Surface Infrastructure

Surface infrastructure may include access roads and rail lines, administrative and industrial buildings, water supplies (dams, pumping from acquifiers), townships, shaft headframes, sewerage treatment facilities, municipal and industrial waste disposal systems, and facilities for the supply of energy. All of these have important geotechnical components in terms of site characterization, design, and construction supervision.

The infrastructure is generally developed on a project basis following a preliminary or final decision by a mining company to develop a new mine or expand existing operations. These project activities may be undertaken under the auspice of either government services, joint development by government and mining companies, or by the mining companies themselves. Irrespective, the performance requirements controlled by the geotechnical designs of such aspects as the foundations for the mill or an uninterrupted water supply to the township and industrial processes are vital to the success of a mining operation.

The geotechnical design approaches for the infrastructure are little different whether they be for a mining project or some other form of development. They tend to be civil engineering oriented in nature, with a strong emphasis on the specific geotechnical disciplines of soil mechanics and groundwater hydrology. The important design and construction components are elegantly discussed in the other papers making up this proceedings volume.

Surface Facilities Subsidiary to Production

The subsidiary facilities and activities may include containment facilities for tailings and waste liquids, temporary and permanent spoil piles and dumps for either the ore or the waste rock and soil, leach pads, and the removal and clearing of waste soil and rock overlying the ore.

Dams and ponds for tailings and liquid disposal have similar performance requirements to dams and ponds constructed from earth and rock for what might be considered more conventional purposes such as municipal water supply or water storage for generating hydroelectric power. Leakage must be relatively minimal, and earth and rock structures need to be stable, particularly in terms of possible piping failure or liquifaction failure in the downstream face or toe. The geotechnical design needs to address these aspects carefully.

Tailings dams may be constructed on a one-off project basis. More normally however, capacity is provided initially for a certain number of years into the future, at which time further construction is undertaken to increase the size of previously constructed facilities or to create new disposal areas. Staged construction of dams, where the height needs to be progressively increased, does not provide for optimum conditions to build stable water-retaining structures. Careful attention needs to be paid by the geotechnical engineer in this regard to design and construction aspects to provide for safe and economic disposal facilities.

Tailings dams, tailings basins, and storage ponds are rarely, if ever, impermeable. Some degree of infiltration of the stored liquids into the groundwater system is inevitable and assessments of possible contamination are required as part of the design, and during the operation of the facilities.

A trend is developing for the geotechnical profession to accept a greater responsibility for the overall management of the disposal systems on a contract basis on behalf of individual mines. Under this system the relationship with the mine tends to progress from one of a project nature to one of an on-going service nature. This trend will provide interesting challenges for the geotechnical profession in the future.

The stability of stockpiles and waste dumps is very important to the mining industry. The material is generally free dumped, resulting in constantly changing geometrical profiles and corresponding dynamic changes in relative stability. Water levels within the dumps or stockpiles change in response to runoff and infiltration from rainfall and snowmelt. Increased water levels invariably translate into reduced stability, as does the influence of vibrations from earthquakes. These are some of the aspects that need to be considered in assisting mines to develop safe and economic disposal and stockpiling practices.

The adoption of poor waste dump practices can, in the extreme, generate catastrophic consequences. For example, in mountainous terrain, there are often economic incentives to dump waste material from some elevated position down natural valley slopes. Often the stability of the mountain slopes themselves is questionable. If waste dump failures occur, large volumes of material may flow over very large distances causing considerable property damage and potential safety problems. Sound design and operating practices are required under these circumstances.

Leaching has become a popular method of extracting metals, particularly gold, from the host rock once it has been excavated and crushed. The rock is heaped onto a leach pad designed to prevent or limit infiltration of liquids into the ground, and sprayed with a leaching agent which is collected and subsequently processed. Leach pads leak; the leaks may be small but are still finite, resulting in loss of leaching agent and potential harmful contamination. The design of the leach pads, supervision of the installation of impermeable or low-permeability liners, and assessments of potential groundwater contamination have developed into important geotechnical disciplines.

The removal of waste soil and rock overlying the orebody, particularly of open pit operations, may be undertaken as an integrated operation with production mining, or as a separate and generally preceding activity. In either case, the volume of material that is removed needs to be kept to a minimum. Thus, it becomes a challenge to the geotechnical engineer, particularly in weak overburden soils containing large quantities of groundwater, to design push-back slopes as steep as possible that remain stable for a range of seasonal climatic conditions. Some of the associated activities that need to be undertakien include construction of diversion channels for rivers and streams, and dredging of soils beneath lakes before draining the lakes.

Production Mining

Most mines use either open pit or underground excavation techniques. The remainder generally rely on solution mining methods or other types of in-situ extraction techniques. There are important geotechnical components to successful in-situ mining, although these are not discussed further in this paper.

The most fundamental question that the geotechnical engineer addresses in production aspects of open pit mining is - 'what is the maximum angle the pit slope walls can be excavated and remain stable?' This effectively translates into - 'what is the minimum stripping ratio to recover the ore?' The techniques available to modify the natural stability of the rock slopes include depressurization of the groundwater in the rock mass, and the use of artificial support such as rock bolts and cable bolts.

The equivalent fundamental question in production underground mining is - 'what is the maximum size of openings that can be mined with out precipitating major instabilities and at the same time achieving high extraction of the resource?' Openings to access the resource are generally in the waste rock hosting the ore. The volume of waste rock removed needs to be the minimum required to permit efficient mining. Underground stability can be improved through well-designed support using rock bolts or cable bolts and the placement of backfill in the voids that have been created.

The evolution of most open pit and underground mining projects is marked by the following milestones:

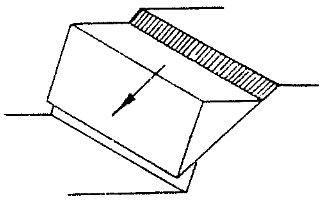
- Exploration identifying the presence of a potential orebody. This phase often involves core drilling;
- Exploration confirming the deposit is of economic interest. This may involve exploratory shafts and drifts and collection of bulk samples;
- Pre-planning including the selection of mining methods and development of general mine plans;
- Pre-stripping (open pit) or excavation of shafts and access/development drifts (underground);
- Production mining.

There is an important geotechnical component to each phase. During the early exploration stages, the data collection program may be minimal, but will become more active and focussed as the economic potential is confirmed. This will lead to the collection of information from which the quality of the rock, the dominant structural geology features, and the ground water conditions are assessed. Initial data are collected from core information and surface outcrop mapping. This transforms into mapping of exposures, determination of the strength of the different rock types, and measurement of groundwater inflows and pressure changes as exploration shafts and drifts are developed.

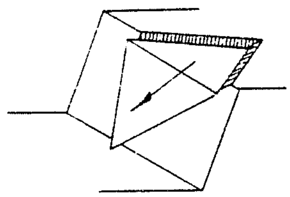
The pre-planning phase involves close liaison between mining and geotechnical engineers to develop an optimization between efficient mining methods and control over stability.

While this may not be an equally weighted partnership in terms of the required level of engineering input, with mining dominating over geotechnical, it is not uncommon for stability to be the controlling aspect in terms of developing the mining approach best suited to the geometrical layout of the orebody and the quality of the rock.

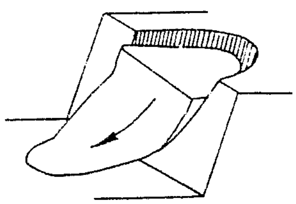
The technical aids available to the geotechnical engineer at this and future stages of the development of an open pit mine include sliding stability analyses of structurally controlled wedges and potential rotational failures of the pit walls (Figure 1), and analyses of the rate of natural or induced ground water depressurization of the walls and floor.



PLANE FAILURE



WEDGE FAILURE

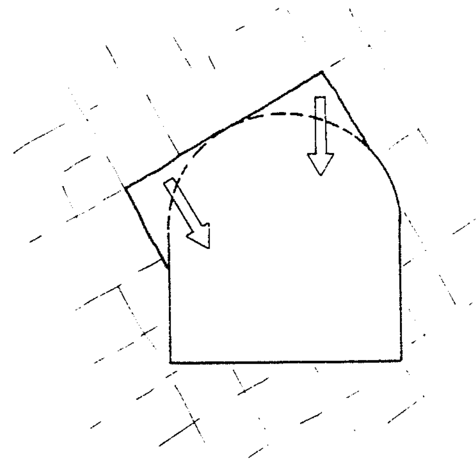


ROTATIONAL SLIDING FAILURE

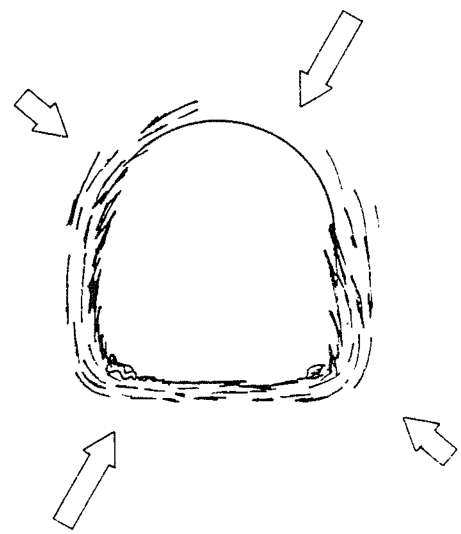
Figure 1
MECHANISMS OF SLOPE INSTABILITIES

The stability of underground excavations is generally controlled by potentially unstable blocks or slabs in the roof or walls defined by structural geological features, and by induced stresses exceeding the strength of the rock mass (Figure 2). Analysis techniques are available to assess stability in terms of these controlling mechanisms. Generally, one of the two mechanisms dominates, and this becomes the controlling approach in assessing the future stability of the mine.

Geotechnical assessments become an on-going part of the development of a mine as it enters the production phase and then becomes a mature operating mine. The minimum level of geotechnical input for most operating mines involves a full-time rock mechanics engineer to assess day-to-day stability aspects, supported by periodic review by an in-house mine services group and/or external specialists.



STRUCTURALLY CONTROLLED FALLING AND SLIDING BLOCKS



STRESS-INDUCED FRACTURING AND SLABBING

Figure 2
MECHANISMS OF UNDERGROUND INSTABILITIES

CRITICAL ISSUES AND TRENDS - OPEN PIT MINING

Geotechnical input to open pit mining reached a degree of maturity during the early and mid stages of the last decade. In earlier times, lack of pit slope design experience was reflected in a somewhat 'snotgun' approach to data collection. The effort expended was often disproportionate to methods available to assimilate and analyze the information. However, as more experience was gained and general conclusions were synthesized from this experience, a practical balance was reached, and a clear separation developed between critical and non-critical design issues.

Since the mid to late 1970's, a clear sign of technical maturity is reflected in the concerted efforts that have been made to present stability assessments in a form that can be easily input into the economic evaluations conducted by mining engineers. There has developed along with this a closer integration of geotechnical and mining activities, with geotechnical engineering being seen less as a service component and more as an integrated part of design practice. Mining engineers have become much more comfortable with recommendations of a geotechnical nature and geotechnical engineers have learnt to present these in terms of economic and safe mining practices.

The economic frailty of the mining industry beginning in the early part of this decade has imposed increased incentives to steepen pit slope walls and still maintain a safe working environment. This challenge is compounded by many pits being excavated to greater depths, with potentially increased stability problems. In response to these challenges, the mining industry has utilized the technical expertise of geotechnical engineers to operate with a reduced margin of reserve stability. This has been achieved through increased and improved monitoring of groundwater pressures and movements of pit slope walls. The results of this monitoring assist in deciding when potentially unstable circumstances pass over the threshold between stable and unstable operating conditions and when alternative mining strategies are required. In other words, the industry is learning to live and operate safely on the brink of pit wall failure, with the knowledge that optimum stability conditions are demonstratably being achieved.

Improved stability is being achieved through improved techniques of depressurizing pit walls and through the installation of artificial support. Improvements in depressurization techniques include more cost-effective drilling of long drainage holes into the pit face, and increased consideration being given to the construction of drainage adits, a technique that was once considered prohibitively expensive in terms of cost-benefit ratio. Long grouted cable bolts are being used to increased benefit to artificially support slopes. Techniques developed in underground mining have been adapted to open pit applications whereby cable bolts are installed before the walls being supported are finally exposed (Figure 3). In this way the face is pre-supported; cables are effectively installed ahead of when they are required. The rock around the penetrating cable ends is subsequently blasted away and any protruding cable ends are cut off and discarded. These techniques which are demonstrating excellent results are the subject of on-going applied research and application by the geotechnical profession.

CRITICAL ISSUES AND TRENDS - UNDERGROUND MINING

Underground mining is often catagorized into hard rock mining, which includes many of the world's base metal and precious metal mines, and soft rock mining, which includes coal, salt and potash mines. Uranium can be found in both hard rock (e.g. mines in Elliot Lake, Ontario) and soft rock (e.g. mines in northern Saskatchewan and New Mexico) environments. Oil mining and oil sand mining generally represent soft rock mining, while oil shale mining can span the spectrum from soft rock through to hard rock mining.

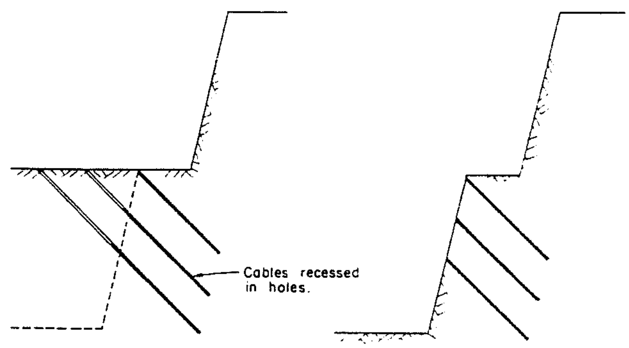


Figure 3
PRE-SUPPORT OF SLOPE WITH CABLE BOLTS

Hard Rock Mining

To varying degrees, hard rock ore bodies tend to be planar and tabular in shape. Individual ore bodies may not be limited to a single contiguous deposit and en echelon tabular deposits are not uncommon.

The underground mining method that is adopted is controlled to a significant degree by the dip of the orebody. Part of this control comes from a difference in the stability of openings in flat dipping and steep dipping mines, and part from the handling of broken rock created by blasting or mechanical excavation under these two conditions. In simple terms, steep dipping ore bodies are those where broken rock will flow naturally under gravity down the dip of the orebody in an excavation mined to the ore contacts. Broken rock in a flat dipping orebody will not flow naturally down the dip of the orebody.

A common objective in most mines is to achieve a very high percentage of extraction of the deposit over the life of the mine. This becomes an important challenge to the geotechnical profession to assist mining engineers to develop mining sequences and layouts whereby controlled stability is maintained and high extraction is achieved.

- Flat Dipping Deposits

Flat dipping deposits are often mined by a room-and-pillar mining method, or some derivative of room-and-pillar mining. The initial geotechnical design decisions relate to the size of the room spans and pillars that will provide for regional stability, safe operating conditions, and economic mining.

The design of roof spans is based, to a significant degree, on experience in other mines in rock of similar quality. The present development of useful analytical techniques is somewhat limited in this regard.

The design of the size of the pillars is based on the stresses imposed on the pillar as mining progresses, compared with the strength of the pillars. One of the main factors affecting the strength is the geometry of the pillars, with tall slender pillars being weaker than short squat pillars. Sophisticated computer analyses have been developed to determine stress distributions in the pillars, and these are used extensively by the geotechnical engineer in assessing optimum pillar sizes.

The question inevitably arises - 'can the pillars be completely or partially extracted after primary mining has been completed?' This introduces a number of topics that need to be addressed by the geotechnical engineer, and changes an otherwise simple task into a more complex procedure. The alternatives are to undertake the primary mining with as high extraction as possible and leave the remaining pillars permanently in place; or to undertake the primary mining with a reduced extraction ratio and then aim for subsequent total or partial extraction of the pillars. The final recommendation of the geotechnical engineer will be based on factors such as the geometry of the orebody, the quality of the rock, and the practicality and economy of backfilling the voids created by the primary mining with sand or waste rock before subsequently extracting or partially extracting the pillars.

Severe stability problems can develop in room-and-pillar mining if mining extends over a large area with a uniform and regular layout of rooms and pillars, and some of the pillars start to fail. If the remaining pillars have very little reserve strength, an unstable propagation of pillar failure can develop, resulting in a 'domino' type collapse over a large area. This can occur during either primary mining or secondary extraction of the pillars. Multiple pillar failure of this type generally occurs quickly, although, as demonstrated in one of the room-and-pillar uranium mines at Elliot Lake, Ontario, the propagation of failure can also occur in a slow progression over several years, ultimately affecting a large area of the mine (Hedley et al., 1984). In this particular case, the release of significant seismic energy was associated with the slow progressive failure of the pillars. The geotechnical engineer needs to be very aware of such potentially dangerous eventualities, particularly in responding to the needs of the mining industry to achieve a high extraction of the resource.

- Steep Dipping Deposits

The most favoured mining methods for steep dipping deposits are cut-and-fill and long hole open stoping. Some of the geotechnical problems that need to be addressed are common to the two methods, while others are specific to the actual method adopted.

Cut-and-fill mining is characterized by small mining increments which are immediately backfilled in a corresponding incremental manner. Hydraulically placed sand is the most common backfill material. Cut-and-fill mining is an 'entry' mining method; personnel maintain physical access to the openings being mined. Hence, the major geotechnical concern is to maintain the stability of the openings to guarantee safe working conditions. This is achieved through designing the size and shape of the mine openings to provide for optimum stability, and designing required rock reinforcement and support such as rock bolts, cable bolts, mesh, and shotcrete.

Long hole open stoping generally involves larger size excavations than cut-and-fill mining. Individual excavations (stopes) are generally separated by pillars and the excavations (typical dimensions, 20 m to 100 m) are often subsequently backfilled. The pillars may then be mined in a following sequence to the primary (first pass) mining.

Long hole open stoping is generally a 'non-entry' mining method where personnel do not physically enter the actual excavations that are created to mine the ore. Safety within the stopes themselves is not of direct geotechnical significance, although it is in the peripheral excavations that provide access to the stopes.

Regional stability of the mine is of major geotechnical significance for both mining methods. This controls the size of the excavations created in the ore, the size of the pillars, the sequence of excavation, and the support and reinforcement that is provided.

Mining by either of the above mining methods is often conducted on intermediate horizons within a mine at the same time. In order to obtain maximum benefits from the gravity flow of broken rock, mining generally advances upward from each intermediate horizon. As a mining horizon on a particular intermediate level advances up towards a mined-out area above, a web of rock is created which becomes progressively smaller with continued mining. This 'crown' pillar may become very highly loaded, ultimately resulting in stress-induced fracturing and slabbing of the rock. This failure process may generate the release of significant seismic energy and 'rock burst' conditions may develop.

Under these somewhat common yet adverse circumstances, detailed geotechnical assessments are required to determine if safe and economic mining can continue in these areas, what type of support or reinforcement is required, and whether alternative mining methods or mining sequences should be adopted.

The increased use of grouted cable bolts and backfill has had a significant impact on improving safety and reducing mining costs over the past 10 to 15 years, particularly in steep dipping hard rock mines. These techniques have been developed for mining applications under the direct guidance of geotechnical engineers experienced in underground mining requirements.

Cable bolts have a very high strength, in excess of 50 tonnes per installation. Because the cables are flexible, they can be installed over long lengths from openings of limited dimensions. Cable bolts perform particularly well in a 'pre-support' mode where they are installed from an initial access opening before a major production excavation is created. The cables may even penetrate through the proposed zone of the excavation and then subsequently be partially mined out as the excavation is created. The remaining portions of cable however provide excellent support to the rock around the excavation.

Mine backfill is generally composed of sand or crushed waste rock. Cement is added to the backfill primarily to be able to expose a vertical free face of fill as part of excavating ore immediately adjacent to fill. This type of sequence is common during pillar recovery in long hole open stope mining. If the exposed fill is unstable, it dilutes the ore, resulting in an increased cost of mining. In cut-and-fill mining, vertical exposures of backfill are not an integral part of the mining method, and cement is only added to a thin surface layer of backfill to improve trafficability for the operation of mine vehicles.

The main challenge to the geotechnical engineer in designing a backfill system is to provide for a stable vertical fill face over the proposed stope height using a minimum proportion of added cement. This will minimize fill costs. Other factors that need to be considered include the rate of drainage of excess water in hydraulically-placed backfill; precautions that need to be taken to prevent excess pore pressures from developing which may cause bulkhead failure and piping of the backfill at the base of the stope; the potential for segregation of the backfill during placement, potentially reducing the strength of the fill; and methods to minimize the volume of water used during fill placement to reduce water handling and other inconvenience costs.

Soft Rock Mining

- Coal Mining

Whether or not to mine using a caving or non-caving system represents an important geotechnical design decision in underground mining of coal seams.

Room and pillar mining is a typical non-caving system. The size of the pillars to provide support to the overlying rock, and the width of rooms to provide for safe economic mining require detailed geotechnical assessment.

A caving system generally involves longwall mining, or room-and-pillar mining followed by subsequent extraction of all or some of the pillars. Longwall mining involves complete extraction of the coal seam within large sized panels, generally using large mechanical excavation equipment. In both cases, the rock overlying the mined-out areas collapses into the mined void soon after the mining front has passed.

If a caving method is adopted, it is important that caving of the overlying material occur close behind the mining face. Dangerous conditions can develop if large metastable hang-ups develop and the rock suddenly releases. Detailed geotechnical assessments are required of the potential cavability of the overlying formations before a mining approach is selected.

There are many other geotechnical issues that need to be assessed in underground coal mining. For example, the size of entry pillars for longwall mining, methods to support the entries, the influences of adjacent clay layers on stability of the roof and floor of excavations, and the resulting surface subsidence all require on-going geotechnical investigation.

- Potash Mining

Potash deposits in Saskatchewan are mined using a room-and-pillar mining method. The potash seams are generally flat dipping, of limited height, and have a large areal extent. The rooms and pillars are generally long and narrow, and large borer machines are used to excavate the ore.

The two deposits in New Brunswick where mining recently commenced are much more irregular in nature than the Saskatchewan deposits and a range of mining approaches are being adopted.

The unique aspects of potash rocks (sylvinite) and the normal host rock for the potash (halite) are that these evaporite rocks are soluble in fresh water, and they undergo creep deformations under an applied deviator stress or principal stress difference, a common stress state around underground excavations. The solubility of the rock dictates that the geotechnical engineer adopts design criteria that ensure groundwater inflows to the mine do not develop. The consequences of inflows developing have been clearly demonstrated by the recent flooding of three potash mines in Saskatchewan. One of these mines was recently abandoned and the other two have incurred major remedial costs.

The stress distributions around excavations in sylvinite and halite rocks are continuously changing as stresses are redistributed under creep action from areas of high principal stress differences to areas of low principal stress difference. Generally, the stresses close to the edge of excavations decrease with time, and those away from an excavation increase. The geotechnical design engineer has to be aware of these fundamental stress transfer processes. Otherwise, excavations may be made in areas of temporarily high stresses that may have subsequently become low stress areas. Naturally low stress areas are more conducive to the creation of stable excavations. The creep properties of evaporite rocks can be used to advantage in mining multiple parallel openings in that the initial excavations may be somewhat unstable, but subsequent openings excavated in close proximity to these can be mined in a more favourable stress environment to maintain stability.

Other Important Practical Aspects

- Standard Practical Design Aspects

The geotechnical aspects discussed for underground mining in the preceding sections are of a general design nature. There are a range of other topics of both a direct and sometimes indirect nature which form part of the geotechnical design process or for which the geotechnical engineer provides direct assistance to the mining industry.

Very little mention has been made of structural geology and yet structural features have a major control on underground stability. This is an area of common interest to the geotechnical engineer and mine geologist. Hence structural geology data collection programs are often successfully integrated with the mine exploration and ore definition programs. These data are used by the geotechnical engineer to assess stabibity problems related to structurally defined blocks falling or sliding from the roof or walls of excavations, or the influence of structure on the strength of pillars or rock surrounding excavations.

In addition to underground openings created for the actual extraction of the ore, most underground mines include two or more shafts and many kilometers of drifts or tunnels which provide access to and facilitate extraction of the ore and its transportation to surface. There is often enough flexibility in locating excavations to have them in rock of maximum quality in order to minimize support requirements. Assessments of this type need to be made by the geotechnical engineer in consultation with the mine planning engineer. Where possible, access drifts should not be located in such close proximity to subsequent stoping operations that stresses around the drifts become significantly more adverse as mining progresses. A common problem in underground mining relates to rock bolts and other support installed in drifts when they are first mined. The selected support may be satisfactory for the stability of a particular drift as a stand-alone excavation, but through lack of appreciation of the significant stress changes that can develop as the excavation of the ore progresses, may not be designed to meet the more adverse conditions that may subsequently develop. This means that on-going rehabilitation of the drifts is required, a costly prospect because supporting openings that have deteriorated is much more difficult than installing adequate support before adverse conditions develop.

The above is an example of adverse interaction between openings. Such interactions can also be beneficial and it is possible to mine an initial excavation in good quality rock and then to mine subsequent excavations in poorer quality rock to advantage by mining in a stress "shadow" created by the first excavation.

Rock bolts are the most common type of support installed in underground mines. There are a range of different types of rock bolts serving different applications depending on the stress conditions imposed on the excavation, the quality of the rock, the size of the excavations, and whether the openings are of a temporary or permanent nature.

The need for geotechnical assessment of the stability of excavations to mine the ore is not just related to safety or the long-term integrity of the mine. Waste or barren rock which falls into stopes being mined and dilutes the ore can have a significant adverse impact on mining costs. In some cases, the potentially unstable waste rock can be supported with rock bolts or cable bolts. The stability can often be improved by adopting careful blasting procedures including accurate drilling of holes, reduced and/or decoupled charges on the boundary of excavations, and the careful design of initiation delays.

As underground mines in Canada become deeper and the extraction ratio of individual deposits increases, stresses in the rock also increase. It is not possible under these circumstances to resist or prevent rock failure and instead it is necessary to design around the failure so that the impact is minimized. This may be achieved by controlling the rate of propogation of failure, predicting areas of potential failure, and adopting alternative mining methods and methods of support where appropriate. Figure 4 presents the results of three-dimensional stress analyses to demonstrate the change of stresses and extent of rock failure that can develop as mining progresses. Significant seismic energy may be released as part of the failure process and one of the roles of the geotechnical engineer is to recommend mining sequences that will promote gradual progressive failure, as opposed to large sudden releases of energy on a more intermittent basis.

The geotechnical design process generally involves data collection and analysis phases followed by performance monitoring. Monitoring invariably includes visual observations, particularly for indications of stress induced fracturing and slabbing, or potentially unstable structural blocks. Depending on the application it may also include displacement measurements using extensometers, convergence measurements, stress measurements and water pressure measurements.

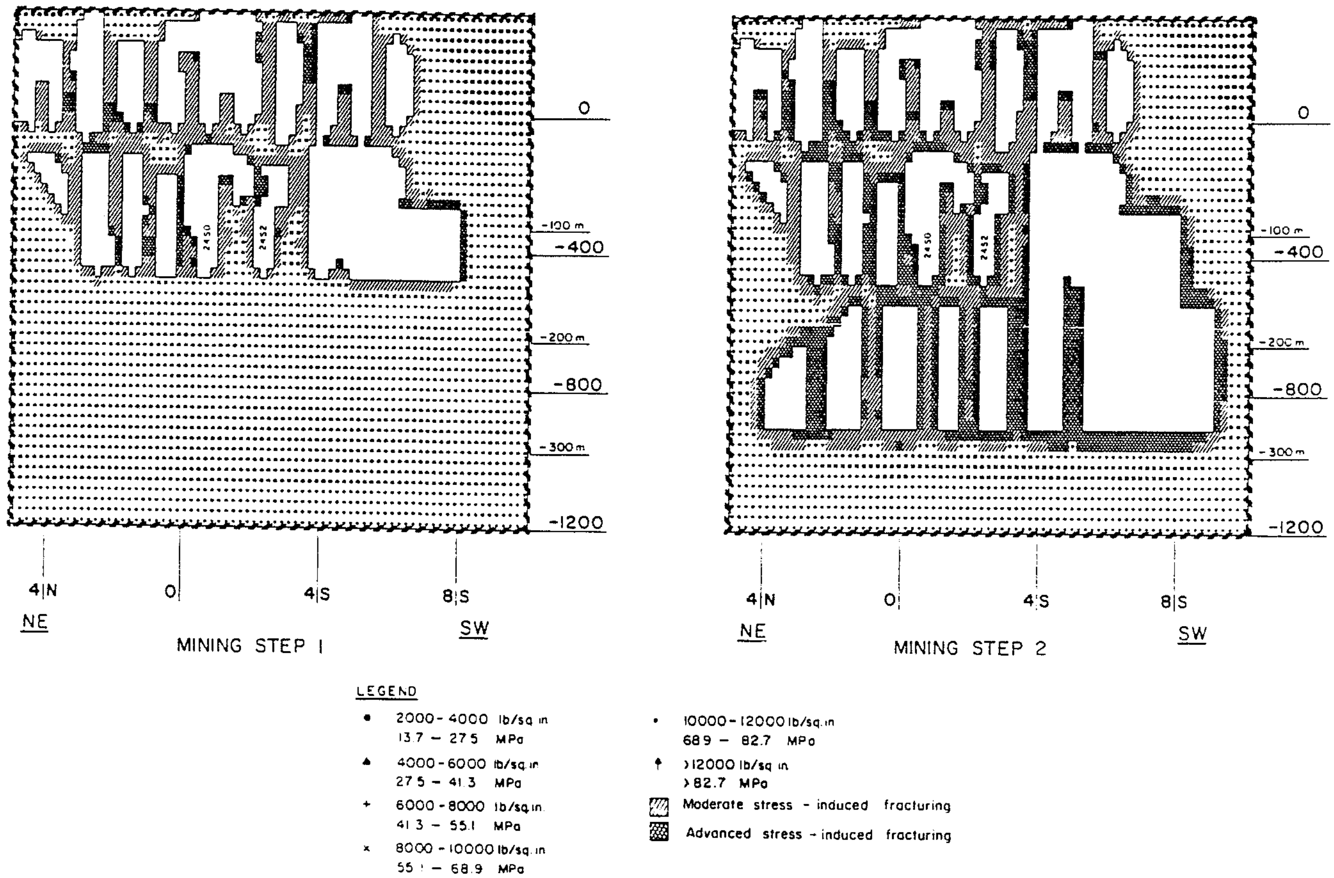


Figure 4

VERTICAL SECTION ALONG STRIKE OF OREBODY SHOWING CHANGE IN STRESS AND STRESS INDUCED FRACTURING

- Novel Design Aspects

Many mines have unique problems to address either because of local climatic influences or particular features of the orebody itself and the quality of the rock. Geotechnical engineers often serve an important role in assessing these novel problems. The following provides examples of the broad nature of some of these problems.

Mine backfill formed by mixing rock aggregate and water, and allowing this to freeze in the stope is being used in a mine in permafrost in northern Canada. Geotechnical studies have been undertaken to determine the optimum aggregate distibution and water content to maximize the strength of the frozen fill. Studies have also been conducted of the use of ice or compacted snow as a backfill material for underground mines.

In some of the uranium deposits presently being explored in northern Saskatchewan for potential underground development, the rock or soil is so weak that freezing techniques may need to be adopted to increase the strength before underground mining activities can proceed. Detailed geotechnical studies are requred before unique activities of this nature are undertaken.

Investigations have been undertaken to recover bitumen from the oil sands of northern Alberta using underground extraction methods. Underground openings would either provide access for in situ recovery (e.g. by steaming), or the oil sand would be physically removed by the excavation process. Detailed geotechnical studies are required to assess the potential for the steaming activities to precipitate safety and stability problems in the shaft and tunnel facilities. Excavating and supporting the oil sand at depths of several hundred meters poses particular stability problems in that the sand matrix is weak, and the multiphase pore fluids consisting of gas, water and bitumen are extremely difficult to depressurize.

Present Status of Practice

Geotechnical engineering applied to underground mining has more recently demonstrated a similar degree of maturity and integration to the established relationship that has developed with the open pit mining industry. Career paths within mines are being established for rock mechanic engineers and geotechnical assessments are being integrated into the mine planning process and the mine safety programs.

CONCLUSIONS

The geotechnical profession provides extensive support to the mining industry in a wide spectrum of activities. Some of these are project related involving the construction of surface facilities required for the general operation of the mine. Others form a direct part of mine planning and production activities for open pit or underground mines. The reliance of the mining industry on high quality geotechnical expertise is increasing, providing an important challenge to the geotechnical profession to assist the Canadian mining industry to remain competitive and adopt innovative solutions to novel problems.

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Geotechnical Practice in Hydro Electric Projects in Canada

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SOMMAIRE

Cette communication porte sur la conception, la construction et la surveillance des barrages en remblai compacté ainsi que sur les méthodes de contrôle des eaux d'infiltration dans l'ouvrage et dans sa fondation. Une courte revue de la mise en place et du contrôle de la qualité des matériaux de remblai, de leur comportement, de l'activité sismique naturelle et induite ainsi que sur l'usage des géotextiles comme matériau nouveau est aussi inclue. Enfin, dans une brève discussion, l'on décrit les initiatives récentes portant sur la sécurité des barrages et la stabilité des talus au pourtour des réservoirs et en aval des ouvrages. Plusieurs exemples sont donnés pour illustrer la pratique canadienne dans le domaine des ouvrages de retenue en terre et en enrochement.

SUMMARY

This paper briefly describes the current trends of design, construction and surveillance of the rolled earth and rockfill dams and their foundations, including the various foundation seepage control measures. A brief review of the fill placement and quality control procedures, performance monitoring, natural and induced seismic activities and use of new material such as geotextile is also given. Finally a short discussion on the recently developed dam safety practice and the stability of reservoir perimeter slopes is provided. Where possible, the design trends are illustrated with examples of Canadian dams.

INTRODUCTION

Thousands of dams of all sizes, shapes and functions have been built in Canada since the early colonization period. Based on the reports presented at the Alberta Dam Safety Seminar (1986), majority of these dams are in Quebec (over 9 000), British Columbia (over 2 000), and Alberta and Saskatchewan (about 1 200 each). Since the construction of the first large dam in 1832 (19 m high masonry arch type Jones Falls Dam in Ontario registered by CANCOLD) followed by the first large embankment dam in 1895 (11 m high Goldstream Dam in B.C.), dam building in Canada has attained the status of a refined technological art and a well developed tradition. The vast knowledge and experience acquired in the design, construction and performance of dams in operation has led to construction of dams of ever increasing size and height and on increasingly difficult and remote sites.

The 1984 CANCOLD Register of Dams in Canada enumerates a total of 613 large dams (as per the ICOLD classification) built to 1983. Of these, 283 dams are earthfill embankment type and 61 of the rockfill embankment type. The trend changed from the mostly concrete gravity, buttress or the arch type dams to the mostly embankment type around 1950. This change of the type of dam coincided with the increase of the height and size of the dams. Recently a significant increase in rockfill dams has also occured. The majority (441) of the large dams have been built for hydroelectric purposes followed by 95 for irrigation and water supply and the remaining 77 for flood control, recreational and other purposes.

This presentation describes the geotechnical aspects of the current practice of design, construction, surveillance and safety organization of the earthfill and rockfill dams and dykes and, to a limited extent, of the auxiliary underground structures. Because of space and time limitations, this text has been confined primarily to the most common trends of design and construction of the rolled earth and rockfill structures. Occasional remarks have also been made regarding the less common but important practices and changes in trends. The authors are well aware of the limitations of this paper in the sense that the examples given herein represent but a small percentage of the numerous interesting and valuable cases available across a country with such vast water resources.

The tailings dams and the hydraulic fill dams being different from the rolled earthfill dams in several respects are not included in this presentation.

GEOTECHNICAL INVESTIGATION

The detailed knowledge of the hydrology, topography and geology of the region is a prerequisite to the feasibility evaluation and comprehensive design of a hydroelectric project. The investigations required to obtain this knowledge constitute major undertakings and involve well coordinated work of a number of experienced and specialized firms and consultants. A majority of these projects being located in the remote northern areas, the investigation methodology and schedules have to be optimized not only from considerations of the economics and the technical requirements but also the climatic conditions, limited period of operation and difficult site accessibility.

The methods used to obtain the necessary geotechnical information on the soil, rock, groundwater and construction material conditions range from air-photo interpretation, walk-over surveys and geophysical surveys to the standard North American exploratory techniques of diamond drilling, sampling and testing. Typical investigations include test pits, trenches, boreholes (with split spoon, Shelby tube and piston sampling, and in-hole standard penetration, shear vane, penetrometer and pressuremeter testing), and standard or especially designed laboratory tests. For exploration at the sites of the powerhouses, galleries and other underground structures, a number of vertical and inclined boreholes is carried out. Some of these boreholes are grouped in a star shaped pattern at a 50° to 60° inclination to ensure an adequate coverage of the rock mass and easy insertion of a geocamera which is used to study and locate the major joint systems and the fracture zones.

Some exploratory wells and/or galleries might also be required at the site of an underground powerhouse to better assess the rock quality through visual inspection, geological cartography and in situ testing.

The extent and nature of investigation is designed to suit the specific needs and prevailing conditions, and the stage (feasibility, site selection, preliminary or final design etc.) of the project. The logistics of camp accomodation, transportation, management of the multidisciplinary activities and the like, constitute a major part of the total cost of investigation for large projects. The mobility of the exploration teams and equipment is essential to a successful and cost efficient operation. Often exploration works begin when land transportation and site accomodation conditions are very difficult or non-existant and manpower and equipment have to be airlifted. These programmes may last several years and are often supplemented just preceeding the construction work when transportation and accomodation become available. The modes of transportation therefore vary from almost exclusively air lifting to the standard ground transportation.

The planning of the investigation programmes aims at striking an optimum or even a compromised balance between the need to obtain sufficient information required for a safe and economical design and the cost of these investigations. Some general criteria which have been used at the James Bay La Grande Phase I Complex for preliminary estimation of the investigation works needed for embankment dams and dykes are summarized as follows:

Embankment Height	Seismic Lines	Number of Boreholes	Number of Test Pits
$H < 8m$	1	1/200	1/50
$8 \le H \le 15m$	2	1/100	1/50
$H > 15m$	3	1/30	1/40

Notes: (1) The number of boreholes and test pits refers to the number required per meter length of the embankment.

(2) The numbers of the boreholes and test pits for the borrow area exploration were respectively 1 per 1 000 m^2 and 5 per 1 000 m^2 of the borrow pit area.

The actual scope of the exploration programmes was determined according to the local conditions and was modified as the work progressed. The geotechnical investigation works carried out from 1971 to 1980 at the different sites of La Grande Phase I are summarized on Table I.

The cost of the above exploratory works, including topographical, hydrological and geotechnical investigations, amounted to some \$ 171 x 10^6, corresponding to 1,6% of the total cost of construction of La Grande Phase I (\$ 11 x 10^9, excluding the cost of the transmission line system) or to 5,6% of the cost of construction of all civil works.

TABLE I

Geotechnical Exploration (1971-1980) - La Grande Phase I Complex

Project characteristics and exploratory works	Project site					
	LG 2	LG 3	LG 4	EOL	Caniapiscau	Total
No. of Dykes	30	70	11	11	93	215
Vol. of Material ($10^6 m^3$)	48,0	35,5	33,4	6,6	34,5	158
Max. Height (m)	162	93	125	33	54	--
Capacity (MW)	5 328	2 304	2 650	--	--	10 282
Boreholes	443	197	201	192	349	1 438
Test Pits	1 170	1 517	980	747	2 236	6 650
Trenches	--	8	51	7	59	125
Seismic Lines (m)	72 600	25 000	37 000	44 200	104 500	237 300
Soundings	322	471	93	471	1 200	2 627

Figure 1 shows the breakdown of the investigation cost for the different activities and indicates the cost of logistics (transportation, camp accommodation and management) to be about 75% of the total cost. If the cost of support activities is distributed among the land surveying, hydrological and geotechnical activities, each of these account for 36%, 18% and 46% respectively of the total investigation cost. These figures illustrate the relative cost of exploration for a large hydroelectric development undertaken in a remote northern area.

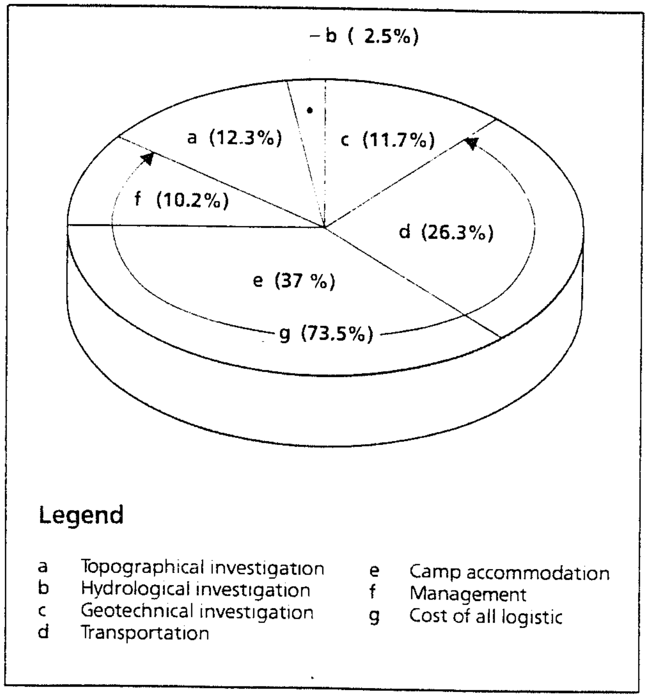


Figure 1 - La Grande Phase 1 - Investigation costs

The foundation investigation for the 183 m high W.A.C. Bennett Dam on Peace River in British Columbia with a fill volume of 63,5 million cubic metres, consisted of geological mapping and about 300 holes of various depths drilled in the river channel and the two abutments (Taylor and Chow, 1976). At the 123,5 m high rockfill type Lower Notch Dam located on a 76 m deep narrow sediment-filled gorge in northeastern Ontario, a total of 188 boreholes yielding 2 514 m in overburden and 2 943 m in bedrock were carried out (Tawil, 1979).

Regarding the underground excavations, some of the interesting examples of the auxiliary structures whose locations and designs have been greatly influenced by the results of geological exploration are the underground powerhouses at Mica, Churchill Falls and LG 2, the surface powerhouse at LG 4, the tunnels at LG 2, and the Downie slide treatment and the diversion portal at Revelstoke. A brief mention of the special features related to La Grande Complex is given hereafter.

The general design philosophy and some innovative design solutions developed at La Grande Phase I have been described by Murphy and Levay (1982). The excellent strength and erosion resistance of the rock led to the use of unlined diversion and tailrace tunnels (over 8 000 m in length and up to 15 x 20 m in section) proportioned to accept velocities of up to 15 m/s.

Based on the results of comprehensive exploration, the location of the LG 2 powerhouse was optimized with respect to a known fault and a suspected shear zone. The potentially costly excavation and support problems were avoided by fitting the main caverns (47 m high x 26 m wide x 483 m long) between the two weak zones. The knowledge of the generally excellent quality rock and favourable disposition of jointing also allowed use of rock ledges to support the powerhouse and surge chamber cranes. Compared to the conventional column and beam alternative, the rock ledges offered the advantages of early availability of cranes for all concreting activities, a more uniform distribution of the very large crane loads in the supporting rock and a reduction in the powerhouse excavation volume. The general arrangement of the LG 2 powerhouse including the intake, penstocks, surge chamber and tailrace galleries is shown on Figure 2.

The design of the LG 4 powerhouse, on the other hand, was changed from an underground powerhouse to a surface powerhouse. The additional costs involved in treating the rock associated with a fault zone located across the site of the proposed underground powerhouse was one of the many factors considered in the decision.

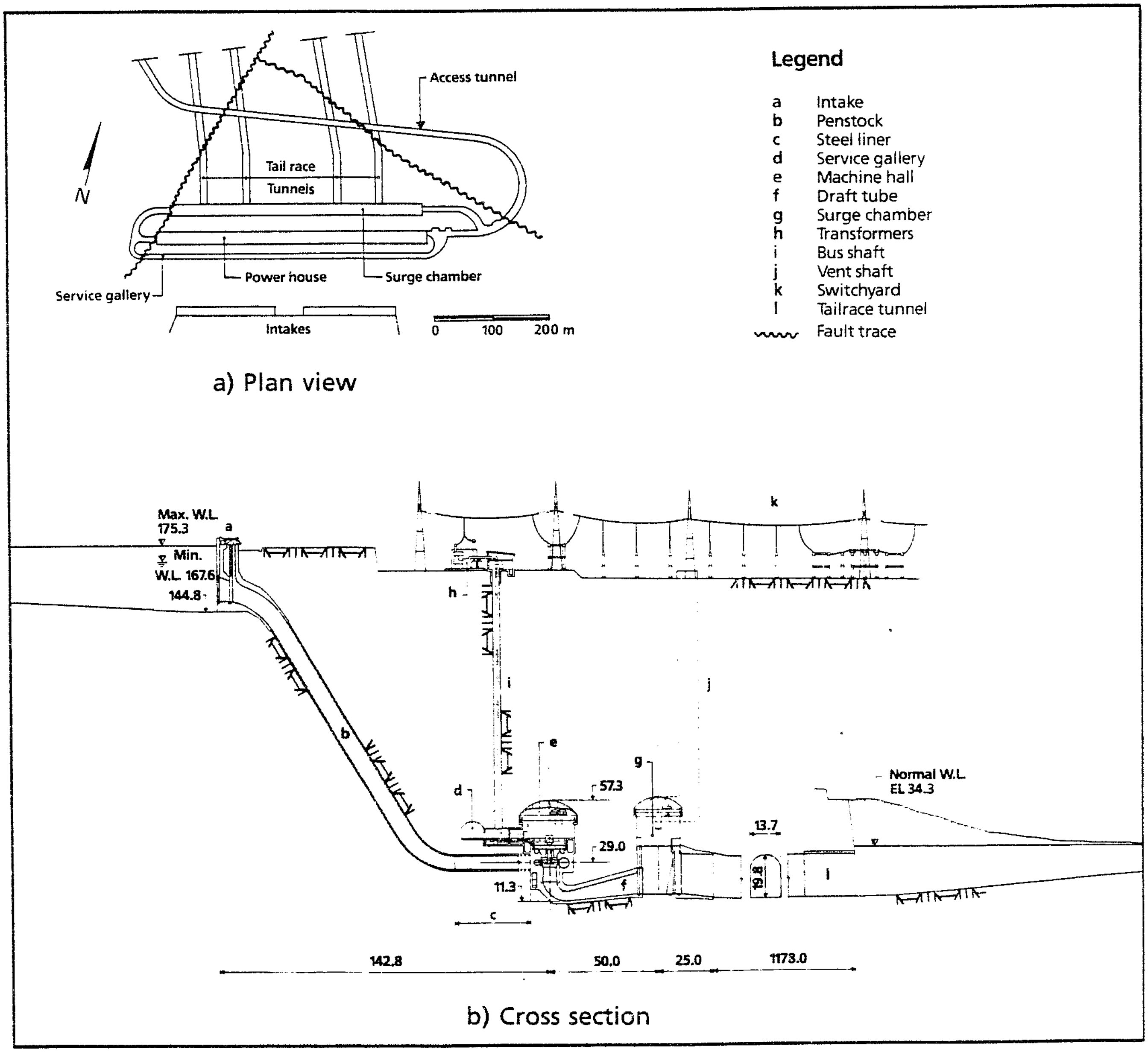


Figure 2 - LG 2 Underground powerhouse

EMBANKMENT DESIGN

Prior to the advent of modern soil mechanics, the earthfill dams were designed primarily by empirical methods. The rapid advancement since the 1940's has led to better understanding of soils behaviour and insights into the performance of dams, and consequently, to the improved principles and techniques of design and to better quality control of construction.

The most important factors controlling, initially the choice of the type of dam and, later its detailed design are: the site geology, the dam foundation conditions, the availability and nature of construction materials and the climatic, topographic and hydraulic conditions. The design of an embankment dam evolves as the works progress and, in fact, the design is not considered "final" until the end of construction of the dam. Some examples illustrating the application of this evolutionary design approach, among numerous others, are: Terzaghi Dam (Terzaghi and Lacroix, 1969), Duncan Dam (Gordon and Duguid, 1970), Arrow Dam (Bazett, 1970) and Gardiner Dam (Jasper and Peters, 1979).

The design of an embankment dam is based on criteria respecting the current trends and selected to meet the specific conditions and requirements of the project. In this regard the criteria for one site can not be applied indiscriminately to the others. The criteria and design details of a large project are generally reviewed by a design board or a review board comprised of highly experienced and recognized experts in the various aspects of the project. The importance of experienced designers and boards can be realized from the poor performance of some dams where their services were not used.

Despite the site specific uniqueness of the design, some common features can be recognized. These design trends are described hereafter.

Embankment Geometry

The three general types of embankment dams are: zoned rockfill dam, zoned earthfill dam and homogeneous dam (Figure 3). The choice for a given site is dictated mostly by economic considerations. The internal zoning details vary significantly from project to project and even from structure to structure on the same project, depending on the quality and quantity of available borrow materials and on the foundation conditions. As a result, a great variety of the above three basic designs are encountered across the country.

The crest width for any type of dam depends on the size, height and type of embankment, practicability of construction, size and type of equipment, possible roadway requirements and the seismicity of the area. The most often encountered width varies between 6 and 12 meters although much larger widths are sometimes adopted to meet special requirements (e.g. at Manicouagan 3 and Mica dams).

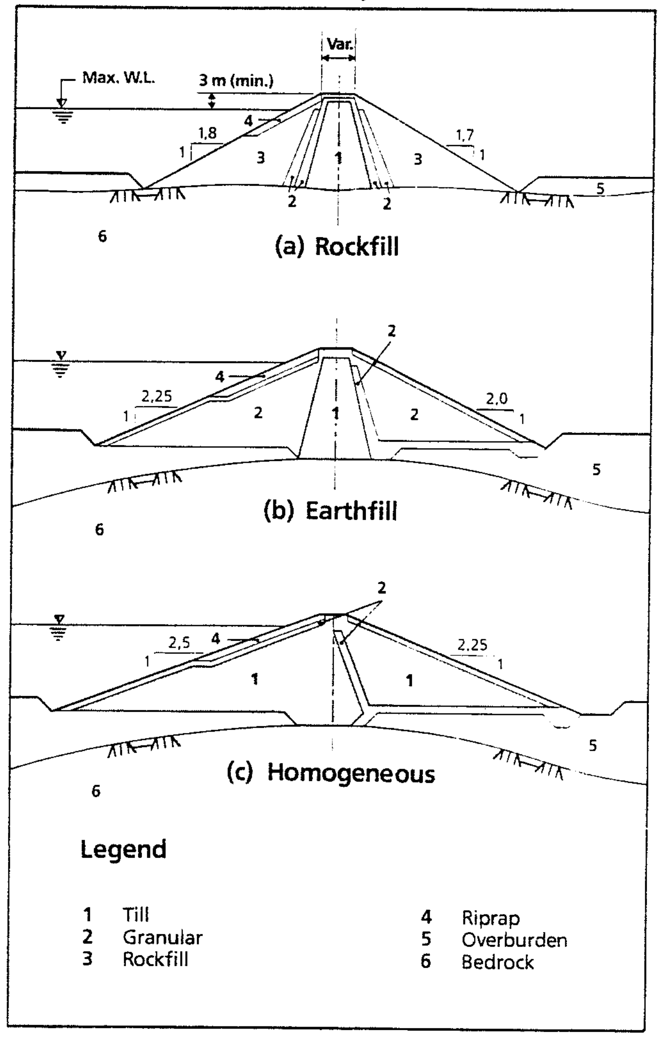


Figure 3 - Typical types of embankment dams

A wide crest is generally achieved by external slope steepening in the upper portion of the dam in order to limit the fill volume increase.

The external slopes of the embankment are controlled by the slope stability and deformation requirements and depend on the characteristics of the shell material and its foundation, as well as the pore water pressure developed within the embankment and its foundation during construction and reservoir exploitation. Stability analyses using appropriate strength and pore pressure parameters are carried out to assess the safety factors against different solicitation conditions including the applicable seismic forces. The most commonly adopted safety factors are:

Conditions	Safety factor
End of construction	1,3
Steady seepage	1,5
Rapid drawdown	1,3
Seismic activity*	1,1

* with full reservoir and steady seepage conditions.

The deformation compatibility of the different embankment zones with one another and with adjacent foundation and abutments is also taken into consideration; this may influence not only the choice of the safety factor but the relative dimensions and inclinaisons of the various zones as well. Dams founded on soft and highly sensitive clay or on bedrock interbedded with low shear strength seams could require higher safety factors depending on the thoroughness of exploration and testing, the environmental conditions and the safety requirements. The dykes and dams enclosing large reservoirs, which can not be emptied readily, are normally provided with multiple lines of defence.

The upstream exterior slope is generally flatter than the downstream slope to account for the greater deformability of a submerged rockfill shell or the drawdown effect on an impervious upstream shell. Some typical upstream and downstream exterior shell slopes of dams located on competent bedrock or dense till foundations with proper drainage facilities in areas of low seismicity are respectively: 1,8:1 and 1,6:1 (rockfill), 2,25:1 and 2,0:1 (sand and gravel), and 2,5:1 and 2,25:1 (till). The exterior slopes on other foundation soil conditions would be flatter. For instance, an 8,0:1 slope for an embankment on soft clay is not uncommon.

Freeboard and Slope Protection

The design of freeboard height is influenced by numerous technical, economical and environmental considerations including the reservoir storage and power generation requirements, the spillway discharge capacity, presence of a fuse dyke and dam safety regulations, and has a considerable bearing on the cost of the project. A detailed treatment of this design element which depends largely on hydrological considerations is beyond the scope of this presentation. Only basic design considerations for the freeboard and slope protection are given hereafter.

The basis for the design is established from wind set up, wave height and wave run up, all of which are functions of the wind velocity and the size, location and orientation of the reservoir. The freeboard is the sum of wind set up and wave run up corresponding to the design wave height (1,25 times the significant wave height for the design wind speed). The design is based either on a one hour extreme storm or on the normal long term continuous wave action during the life (50 years) of the embankment. Both of these approaches have been used in Canadian practice and generally give design wave heights of similar order of magnitude. Final choice is based on local experience and requirements. From a field performance survey of 62 dams in Prairie Provinces, Peters and Towle (1979) have identified thin riprap layers, uniform rock size and use of straight line fetch on wide or circular reservoir as the main causes of unsatisfactory conditions. They have recommanded use of effective fetch, wave height based on special wind studies (which for Prairie Provinces amounted to a 37 year return maximum hourly wind velocity) and rational design procedures.

The final dam crest elevation is established at a level corresponding to the maximum reservoir operating level plus the computed freeboard and an adequate allowance for the predicted embankment and foundation settlement, the camber and the potential crest damage (due to runoff, frost action, etc.). Any possible crest subsidence due to seismic action should also be taken into consideration.

The wave run down is the lowest level on the dam slope which becomes exposed between successive waves and equals the design wave height multiplied by a run down factor. The elevation of the bottom of the riprap is determined from the calculated wave run down plus generally 1,5 m, below the design low operating level of the reservoir.

The conventional types of the upstream slope protection are dumped or hand placed rock riprap, concrete paving, asphalt lining, soil cement and precast concrete blocks. In view of the ready availability of high quality rock, excellent performance record (U.S.B.R., 1977), economy of construction, and relatively easy and not too expensive maintenance, the dumped stone riprap is the most popular type of protection used in large embankment dams in Canada. The material needed for rock riprap is obtained as a by-product of required excavations and/or from quarries. Depending on the nature of the shell material, a riprap bedding and one or more transition zones may be required. The gradations of the various zones of protection should be filter compatible with respect to one another. The total thickness of the bedding layer generally varies between 300 mm to 900 mm or can be taken equal to 1/2 to 2/3 of the riprap layer thickness.

The most important design requirements for the riprap material are their resistance to the Canadian weathering conditions and their resistance to dislodging by wave and ice action. The riprap must be well graded from rock spalls to the maximum size. The standard requirements, normally given by weight, of the maximum (W_{max}) and minimum (W_{min}) size stones relative to the weight of the average size (W_{50}) stone are: $W_{max} = 4\ W_{50}$ and $W_{min} = W_{50}/4$ respectively. The multiplying factors sometimes vary depending, in part, on the anticipated frequency and direction of exposure to the design wave.

The determination of the average weight of stone (W_{50}) from the design wave height is made using the Hudson's or the Bertram's formula as reported by Taylor (1973). The thickness of the riprap layer should be sufficient to accomodate the size of the stone necessary to resist wave action and is generally taken equal to 1,5 times the size of the average stone weight (W_{50}) or equal to the size of the W_{max} rock particle.

The maximum particle size of bedding is generally taken equal to the size corresponding to W_{min} of the riprap. At Churchill Falls, the bedding material consisted of small-size stones and spalls produced by scalping the riprap quarry run material by mean of a grizzly with a bar spacing ranging from 300 to 450 mm (McConnell et al, 1973).

The downstream slope protection in homogeneous or zoned earthfill dams is provided by a layer of oversize gravels, cobbles, rockfill or tunnel muck which should be wide enough to remain stable under the action of running water. No filter layer or bedding is generally required under this protective blanket. Best results are generally obtained when this outer zone is placed simultaneously with the adjacent shell zone and when its width equals the full width of the construction equipment, such that the protection zone can be compacted properly.

Internal Zoning

This section deals with typical internal zoning designs adopted for the zoned earthfill and rockfill dams built in Canada, particulary in the last 25 years. The choice of the number, dimensions and arrangment of the various zones in an embankment dam depends on numerous factors, such as the availability and characteristics of construction materials, the foundation conditions, the geometry of the river valley and abutments, the necessity of material processing, and the risks of embankment cracking, internal erosion and piping. Some interesting examples of the typical design sections of zoned earth and rockfill dams built across Canada (Arrow Dam, W.A.C. Bennett Dam, Duncan Dam, LG 3 North Dam, LG 4 Main Dam and OA 11 Dam) are show on Figures 4 to 9.

The current trends of designs of the core, filter, transition and shell zones as well as the geotextile filters are described hereafter.

Core

Traditionnally referred to as the impervious core, the embankment dam core is constructed of material which is impervious relative to the other zones, which allows only controlled seepage to occur and which remains stable under operating hydrodynamic and deformation conditions. With typical permeability coefficients of 10^{-7} to 10^{-9} m/sec for materials used in dam core whose stability is ensured by controlled gradients and suitable zoning, the overall seepage is seldom considered a matter of concern.

The core in a zoned dam can be either vertical or sloping in the upstream direction, depending on the foundation topography and geology. The width of the impervious core depends primarily on the nature and quantities of the available construction materials as wells as on the compressibility of the foundation stratum and the height of the dam.

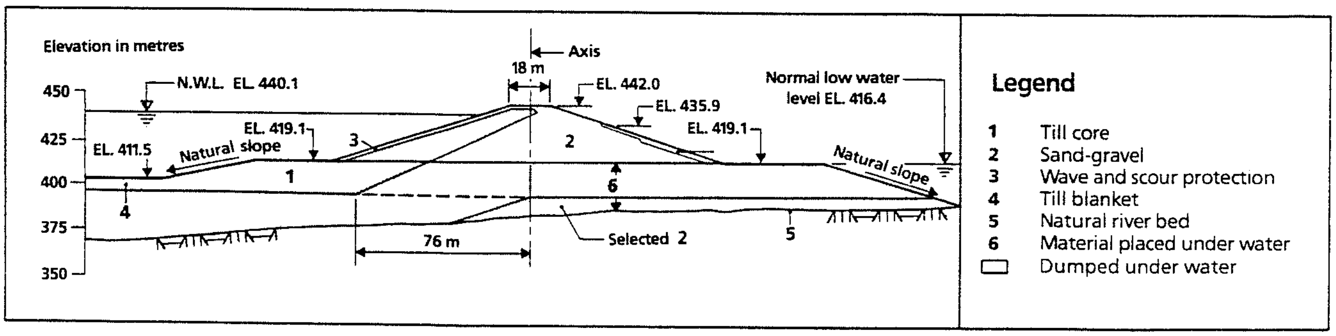


Figure 4 - Arrow Dam - Section

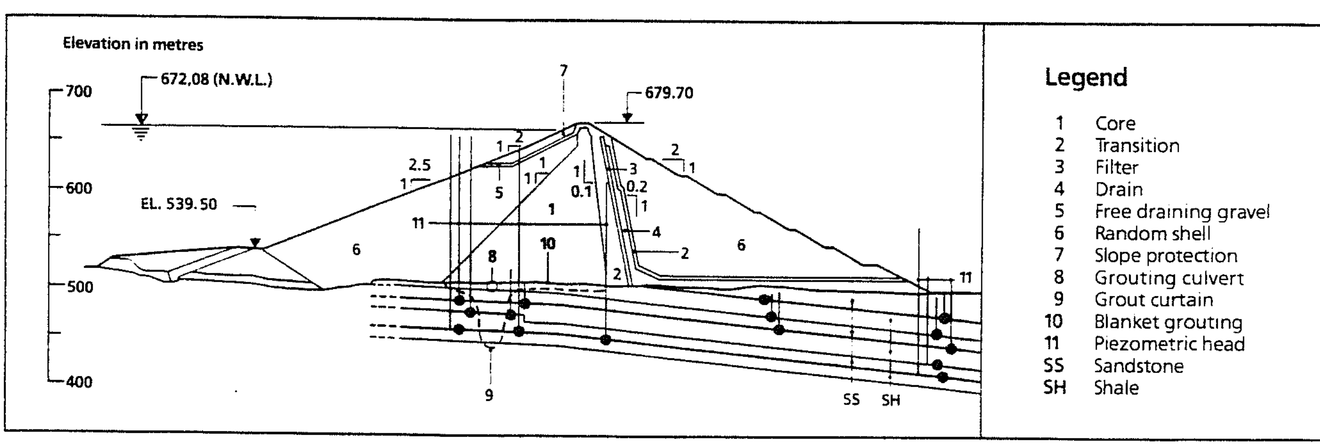


Figure 5 - W.A.C. Bennett Dam - Section

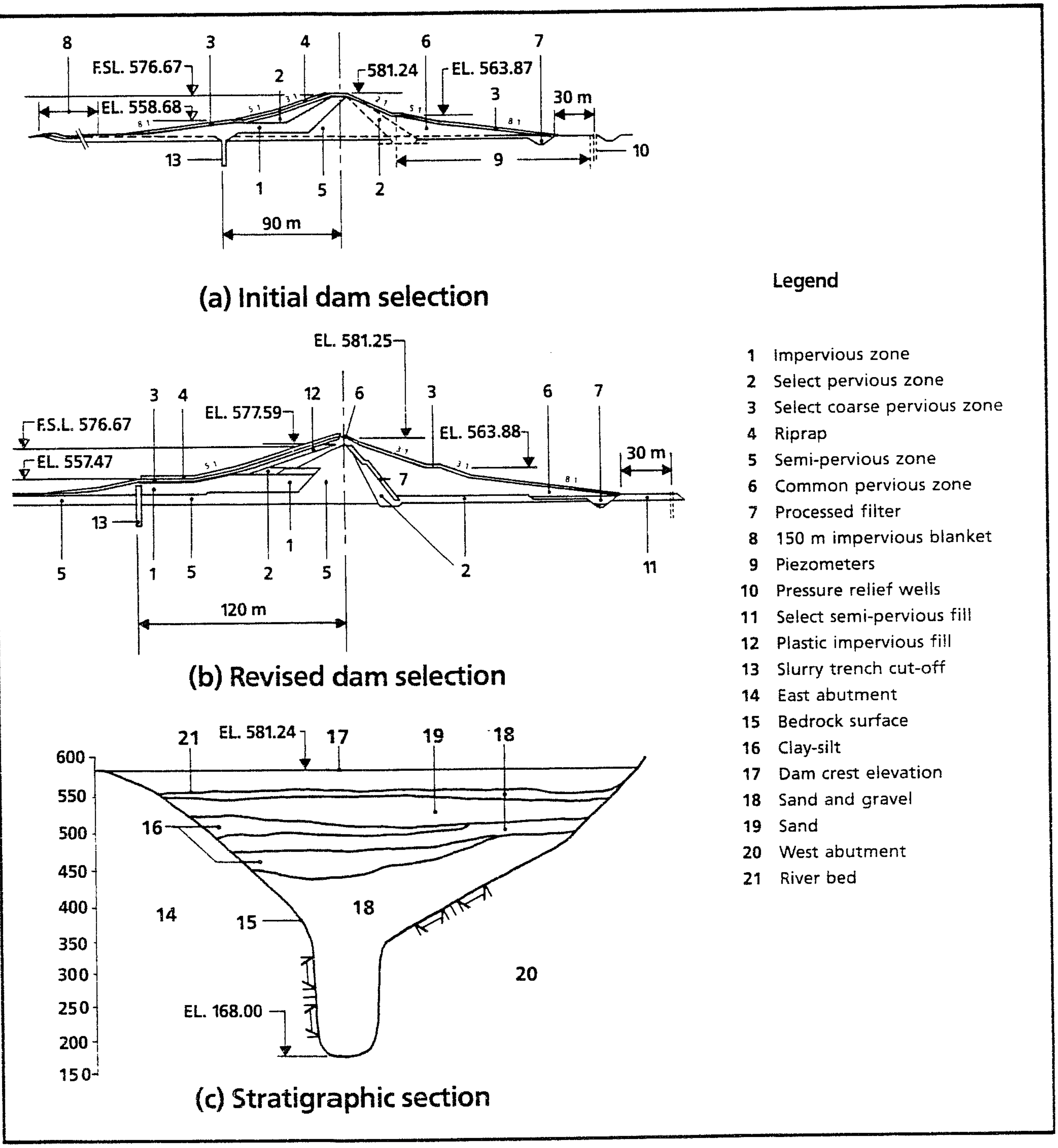


Figure 6 - Duncan Dam - Section

Sherard et al (1963) note that a thin core (15 to 20% of the water head) should have a higher piping resistance and more adequately designed and constructed filter zones than a wide core (30 to 50% of the water head).

The most common core material in Canadian dams is the glacial till which is encountered generally in abundant quantities as a moderately cohesive to non-cohesive broadly graded material with a low compressibility and a high shear strength. Because of the abundance, relative ease of placement and excellent behaviour, the practice has been to use as much of this material as possible and to select relatively wide till cores (widths varying generally between 35 and 60% of the water head) in conformity with economic and technical considerations. As the technical advantages generally outweigh the additional

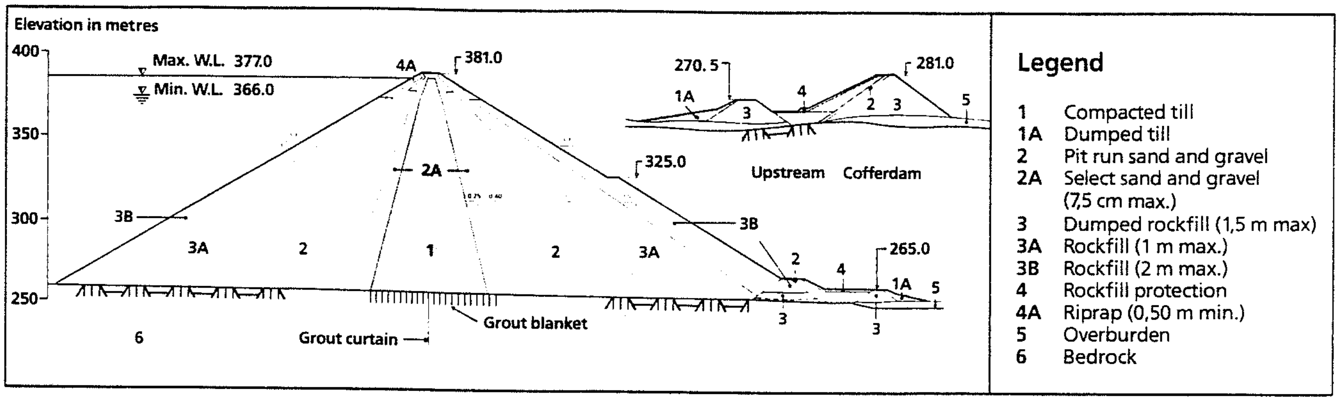


Figure 7 - LG 4 Main Dam - Section

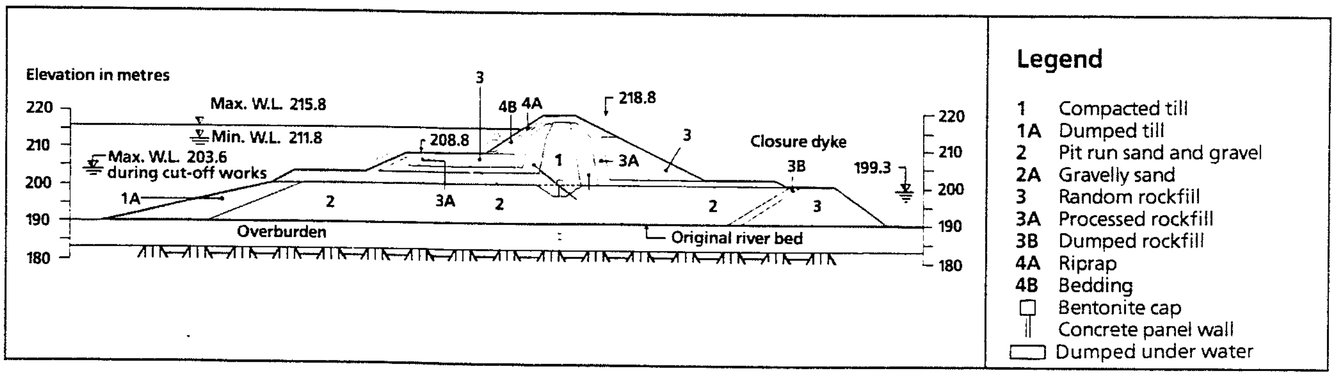


Figure 8 - OA-11 Dam - Section

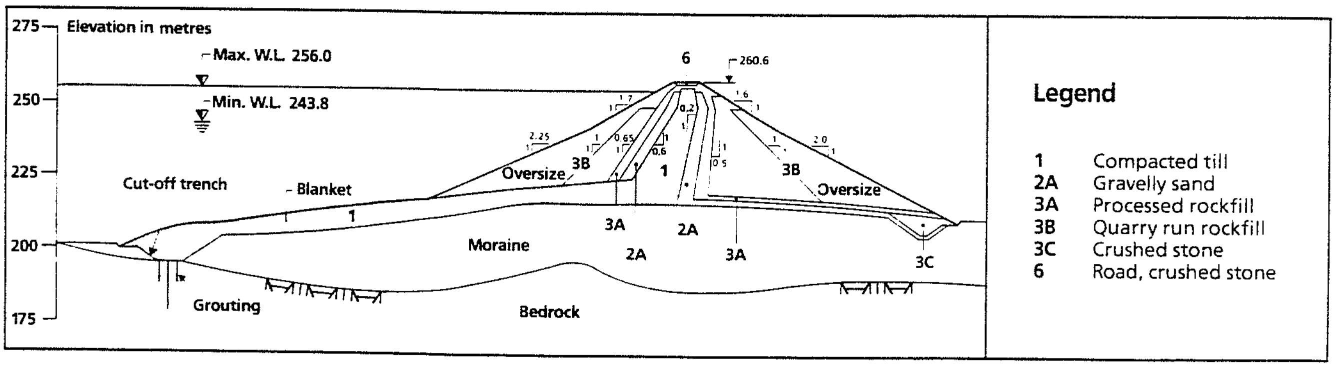


Figure 9 - LG 3 North Dam - Section on right bank

cost involved in providing a wide core, some extra width is considered desirable provided the cost increases are reasonable and acceptable. The relatively narrow widths are employed in cores with slight upstream inclination and wider cores in dams with central symmetrical cores.

The widths of the till cores of the Outardes 4 Main Dam (rockfill dam with slighly inclined core) and the Manicouagan 3 Dam (earthfill dam with central core) were about 30% and 90% respectively. The extra wide core at Manicouagan 3 Dam was adopted in consideration of the presence under the core of a rigid panel wall system installed through a very thick overburden foundation (Figure 10). Generally, dams in British Columbia are also built with thick cores in order to provide greater resistance to piping and cracking and to account for high seismicity (Figure 5).

The top of the core is generally maintained at least one meter above the maximum reservoir level to prevent seepage flow over the core for the full reservoir and wind driven wave conditions. Typically, the top of the core is at least 3 m wide and its surface is sloped upstream to drain the rain

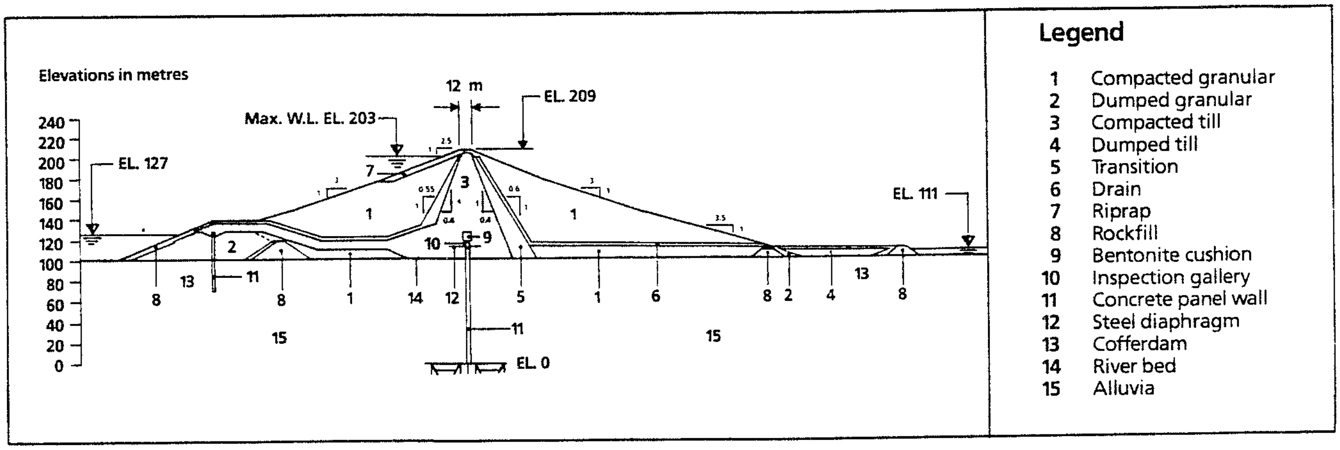


Figure 10 - Manicouagan 3 - Section

and snow melt water into the reservoir. To minimize damage from frost action, the core is protected by creating enough overburden weight and confinement with frost-free fill. At La Grande Phase I, a 2 meter thick sand and gravel cover producing a surcharge pressure of about 60 kPa in the core at the reservoir level has been considered adequate (Paré et al, 1978).

Filter and Transition Zones

The use of properly designed and constructed drainage and filtering components (filter and transition zones, chimney drains, pervious blankets and drain outlets) within the embankment constitutes the first and foremost line of defence against internal erosion. The increased awareness of the susceptibility of the embankment dam cores and foundations to hydraulic fracturing and internal erosion and of the risks involved in the event of failure of the dams of ever increasing size has placed great reliance and importance on the filtering and drainage features. Some of the serious and costly incidents which have occured in the world in the past 40 years could have been avoided if their filter and transition zones had been well designed and constructed. Given such importance, a certain degree of conservatism in their design and construction would be justified.

A single filter zone or multiple filter-transition zones may be required between the impervious core and outer shell zones depending on their relative gradations. Properly designed zoned rockfill dams are provided with transition zones between sandy filter and random rockfill shell zones as well as between overburden foundation and rockfill shell. Multi-layered filters and drains might be required to prevent piping where high discharge capacity is needed for rapid drainage (e.g. in seismic areas).

The filter design criteria, aimed at preventing any migration of soil particles from the base material into the protective adjacent zone while permitting the free drainage of the water seeping through the base material, have been developed from laboratory research and have been used conventionally. The filter criteria were developed for a uniform sand by Terzaghi and confirmed through extensive laboratory testing by Bertram. The most accepted filter design criteria are stated in Table II.

Table II

Filter design criteria (USBR)

$$\frac{D_{15} \text{ of filter material (max.*)}}{d_{85} \text{ of base material (min.*)}} \leq 5 \text{ : to ensure stability against particle migration}$$

$$\frac{D_{50} \text{ of filter material (max.*)}}{d_{50} \text{ of base material (min.*)}} \leq 25 \text{ : " " "}$$

and

$$\frac{D_{15} \text{ of filter material (min.*)}}{d_{15} \text{ of base material (max.*)}} \geq 5 \text{ : to ensure adequate drainage}$$

* Coarse and fine limits of the specified gradation envelopes.

On large projects, the applicability of the above empirical rules is generally confirmed by large scale permeameter tests performed on the exploited borrow materials (Paré et al, 1982a).

The following accompanying rules are generally implemented to realize the implicit spirit of the above design criteria:

a) For broadly graded core materials the filter design should be based on the minus 5 mm portion of the base material.

b) To ensure adequate permeability, the maximum percentage of fines (minus 0,080 mm) in filters, transitions and drains is limited to 5% (generally less than 2 or 3%).

c) The filter, transition and drain materials should be graded from coarse to fine with no gap or excess of any particular size.

d) To minimize segregation during placement, the maximum particle sizes for the filter and transition zones are limited to 75 mm and 150 mm respectively. Some authors recommand a maximum particle size of 20 mm for filter material against clayey cores and therefore consider concrete sand as an excellent filter material.

Subsequent to some dam incidents, the filter design criteria has acquired renewed interest (Gordon and Duguid 1970, Webster 1970, Sherard 1970, 1979 and 1985, and Burke, 1979). Recent findings and reviews (Lafleur 1984, Kenney et al 1985, Kenney and Lau 1985, and Morgenstern 1986) have provided interesting insights into this complex problem. To eliminate all possibilities of particle migration for cohesionless bases and filters with uniformity coefficients of less than 6, Kenney has proposed the use of the coarser of the following controlling constriction size requirements:

$$D_{5F}/d_{50B} \le 4$$

or $$D_{15F}/d_{50B} \le 5$$

It is generally considered that Kenney's criteria, based on severe test conditions of simultaneously applied high hydraulic gradients and vibratory forces, are overly conservative. Nevertheless, Kenney's experimental charts (Figure 11) are considered to have demonstrated the adequacy of the currently used criteria, particularly for the commonly operating hydraulic gradients (Morgenstern, 1986).

Ripley (1986) has emphasized the successful performance of sand filters in the past and stresses that the filter zone material should have:

1) a particle size gradation with controlling constriction size that is appropriate in relation to the adjoining upstream zone;

2) low to nil susceptibility to segregation during placement; and

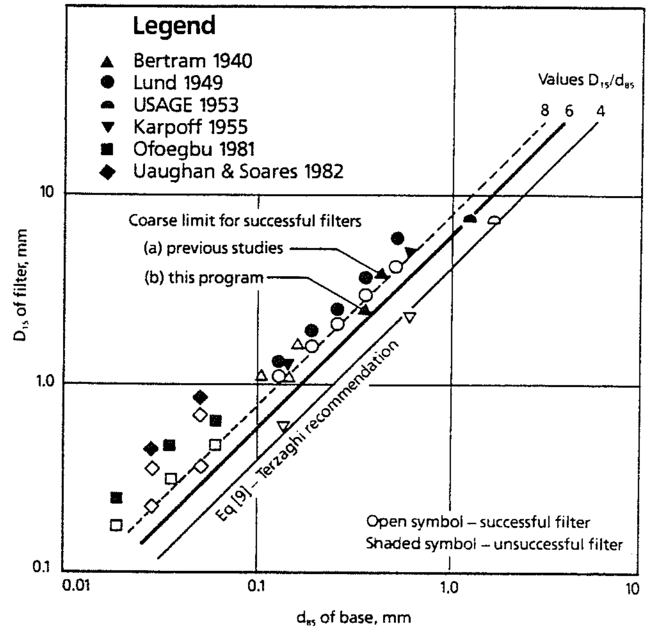


Figure 11 - Results of filter tests (from Kenney 1985)

3) "crack stopper" capability, i.e., the filter material should be incapable of sustaining an open crack within the filter zone as a downstream extension of a transverse crack through the more cohesive core.

Based on a comprehensive investigation by the U.S. Conservation Service into the capability of downstream filters in preventing erosion during concentrated leaks in dam cores, Sherard and Dunnigan (1985) have made the recommandations summarized in Table III.

These requirements indicate the adequacy of the filter zones designed according to the currently accepted criteria in controlling and sealing concentrated leaks.

The filter zones placed downstream of the dam core may consist of natural gravelly sands or crushed stone or a convenient mixture of the two, although natural materials are preferred. A more conservative approach would be required for filter zones constituted of crushed (angular) materials. The filter material, placed upstream of the impervious core, must be able to act as a "crack filler" in the event of fractures and leakage channels developing through the core, particularly in the case of thin cores of rockfill dams. Natural gravelly sand having rounded or sub-rounded particles would be better suited to fulfill such a function.

It should be noted that the stability and reliability of a filter would depend, among other factors, on its thickness which would counteract the inevitable local deficiencies, such as natural heterogeneity of borrow pit

Table III

Filter criteria - U.S. Conservation Service (1985)

	Base soil	Fine content	Criteria
1.	Fine silts and clays	85-100	$D_{15F}/d_{85B} \leq 9$
2.	Sandy silts, sandy clays & clayey sands	45-85	D15F ≤ 0,7 mm
3.	Sandy gravels	0-15	$D_{15F}/d_{85B} \leq 4$
4.	Impervious between groups 2 and 3	15-40	Intermediate between groups 2 and 3

material and segregation induced during placement. Twice the width of the roller is considered a sound average width for the filter zone of intermediate height dams. The width of filter and transition zones in high dams is generally increased with depth and extra width is often provided by flaring against concrete structures and steep rock abutments. Where concrete structures end a short distance downstream of the core and within the filter-transition zones, the latter are widened and wrapped around the concrete structure (e.g. Churchill Falls Project in Labrador; Seemel and Paré, 1979).

Shell

The outermost zones in embankment dams consist of the most commonly available and relatively free draining pit run materials characterized by a high degree of inherent stability under static and dynamic solicitations. Where possible, through selective exploitation of the borrow pits or other such means, the relatively fine material is placed towards the filter zones and the coarser material towards the outer slopes. Thus the practice in rockfill dams build at La Grande Phase I has been to set up an internal shell zone (limited by a 45° sloping line starting from the crest) in which the max. 1 m size rockfill is placed and compacted in one meter thick lifts while max. 2 m size random rock is placed in the external shell zone compacted in 2 meter thick lifts. At Outardes 4 Dam the outer shell zone (beyond the 1,5H:1V line from the crest) was constructed of dumped rockfill material (Dusseault et al, 1970).

The W.A.C. Bennett Dam (Figure 5) on Peace River in British Columbia is a unique example of an earthfill dam in Canada in which the materials for the different zones were obtained by selective processing of the naturally occuring silty sands. Other important examples of earthfill dams with sand and gravel shells are: Mica Dam in British Columbia and LG 4 Main Dam in Quebec. The rockfill shells in the initial design of Mica Dam were replaced by sand and gravel shells in consideration of the weakness under high stresses of the available micaceous rockfill material. The LG 4 Main Dam (Figure 7) consists of an inner shell of sand and gravel and an outer shell of rockfill obtained from required excavations and is a good example of optimization of material costs and embankment volume.

Geotextile Filters

The use of geotextiles in dam engineering has increased, although cautiously in recent years. They have been used primarily as separation layers between zones of dissimilar materials, and to a lesser degree as filtering and drainage media. Their applications have thus been limited to non-critical or temporary functions, such as, filter zones in cofferdams and experimental back-up filters in small dykes.

The reluctance of dam designers to use geotextile as permanent means for filtering and drainage stems from the lack of precedent and long-term performance. The long-term durability and permanence of their hydraulic and mechanical characteristics must be well established before they can be adopted to serve as permanent and essential zones. Their performance under conditions of sustained high stress concentration and large deformation (in contact with coarse angular particles) requires careful study. Concern is often expressed regarding the performance of the overlapping construction joints.

At La Grande Phase I geotextile has been used in cofferdams and under the upstream rockfill shells founded on till deposits. Only the thick varieties of the membrane with a coefficient of normal permeability in the compressed state of at least 100 times the permeability of the base soil have been used. The experience in the cofferdams to date has been satisfactory except at LG 1, where poor overlaps and sheet separations were observed locally at the construction joints when the geotextile installed in the upper part of the embankment was exposed, at time of raising the cofferdam five years after initial construction.

Homogeneous Embankments

A homogeneous dam, constructed mainly with a single material of a fine-grained nature, incorporates an internal drainage system consisting of a chimney drain (within the middle third of the embankment) which is connected to a horizontal drainage blanket and, often, a toe drain (Figure 3). The design requirements of the drainage system depend on the characteristics of the embankment materials as well as the embankment height and the nature of its foundation. Due to pore pressure build-up during construction and the difficulties associated with placement of fine-grained materials in winter, the homogeneous dam height is generally limited to about 30 meters.

To approach as closely as possible the characteristics of a zoned dam, the relatively coarser and pervious materials are placed, where possible, in the outer part of the embankment. The portion of the embankment upstream of the chimney drain is constructed of a relatively finer, wetter and denser material than the downstream portion. The selection of material for relative zoning is generally achieved through selective and careful exploitation of borrow areas.

Both, the upstream and downstream external slopes of a homogeneous dam are generally flatter than the slopes of a zoned dam. The surfaces are protected with coarse materials against excessive frost action, erosion and burrowing animals.

Material Placement and Field Quality Control

The uninterrupted and long term effectiveness of the various embankment zones requires that (i) the seepage through the core, foundation and abutments be fully controlled at all times, (ii) the internal zones be mutually compatible from a standpoint of erosion prevention and drainage, (iii) the total and differential deformation be within acceptable design limits, and (iv) the fill materials, as placed in the zoned embankments, be sufficiently homogeneous and dense to effectively and permanently fulfill their specific design functions.

Traditionally these objectives for the various embankment zones have been achieved by controlling their gradations and their placement density and water content conditions. As compaction procedure and equipment affect the quality and cost of material placement, specific requirements are made regarding the roller type, lift thickness, number of passes and the like. Some of the current requirements are given in Table IV.

The lift thickness over the bedrock foundation and against steep abutment or concrete structures is generally reduced by about 50% to get a better interface contact.

Test embankments are often built at the early construction stages of major dam projects to select the appropriate type of compaction equipment, to optimise the lift thickness and the number of roller passes, and to investigate the feasibility of adjusting the placement requirements to the existing natural moisture content of borrow materials.

The flexibility to deform without cracking under high embankment stresses is often a desirable quality of the core materials and is achieved by compacting them at slightly higher than optimum water contents (Gordon and Duguid, 1970, McConnell et al, 1982, and Paré et al, 1984). However, at water contents exceeding the optimum by a few percent points, the material becomes very difficult to compact and has to be partially dried. The water content may be reduced by suitable handling (exploitation, stockpiling, transporting, spreading and mixing) if ambient air is sufficiently dry. A very effective way to reduce the humidity, however, is by means of a rotary kiln dryer. Use of dryers provides the additional advantages of permitting placement of the heated material during the night, under rainy conditions or during early freezing periods, and extending the placement season by about one month in northern Quebec. Some factual information related to their use at the La Grande Phase I (Paré et al, 1982) and Manicouagan 3 (Bouliane and Reid, 1976) sites is given hereafter.

About one quarter of the core material required for the LG 3 Main Dam was processed in a 420 t/h capacity, 4.2 m diameter and 24 m long rotary kiln dryer, as summarized in Table V.

Table IV

Material compaction requirements

Material (Zone)	Lift Thickness(m)	Roller Type	No. of Passes
Clay (Core)	0,25	sheepsfoot	6
Till (Core)	0,5	pneumatic - 50 t	4
Gravelly Sand (Filter)	0,5	vibratory - 5 t	3
Crushed Stone (Transition)	0,5	vibratory - 5 t	3
Sand & Gravel (Shell)	0,5	vibratory - 5 t	4
Rockfill (Shell)	1,0	vibratory - 10 t	4
Rockfill (Riprap)	2-3	dumped	-

Table V

LG 3 - Processing of core material through the rotary kiln dryer

Operation			Processed Quantity (m^3)	Average Production (m^3/h)	Fuel Consumption (l/m^3)	Water Content (%)	
Year	Days	Hours				Initial	Final
1977	132	768	193 000	251	6,1	9,9	8,4
1978	171	1 458	299 000	205	6,6	10,3	8,6
1979	157	1 283	233 500	182	10,9	11,3	8,5
1980	182	1 387	274 000	198	9,1	11,3	8,5

Two dryers were in operation during the four year construction period at LG 2. During 1978 alone, about 560 000 m^3 of wet till amounting to about one-third of the core material was processed for a total of 3 730 hours of kiln operation. Using a similar dryer, the average annual and the maximum and minimum daily productions at Manicouagan 3 were about 200, 320 and 105 m^3/h respectively; the corresponding fuel consumption varied between 7.4 and 11.3 l/m^3 depending on the natural water content of the till.

The opposite situation is sometimes reported in drier areas of the country where the core material has to be wetted by spraying, mixed, scarified, graded and compacted to get satisfactory results. All these operations require continuous control to ensure construction of homogeneous fill.

The purpose of quality control is to assure that the material characteristics and the placement procedures are in agreement with the technical specifications. The quality control works constitute the most important activity during construction of a dam as the integrity of the structure depends on the thoroughness of quality control. The construction period is the only time when the condition of the foundation and fill can be directly observed and corrected. The control of material characteristics, handling and processing as well as placement equipment and operations, is performed systematically at the borrow area and on the dam. These activities are optimized to allow adequate control and orchestrated with the construction activities and speed of operations. Visual inspection to control borrow material, equipment, procedure and product is of utmost importance as it permits a rapid and general evaluation of the embankment homogeneity and the overall quality of the work. Survey crews and field laboratories provide the necessary technical support to the inspection units.

The following three categories of control tests are generally performed:

a) grain-size analyses and water content determination for a quick verification of the conformity of materials to specifications;

b) field density measurements related to a reference or a target density to monitor embankment density and uniformity;

c) determination of mechanical characteristics, like permeability, compressibility and strength of the embankment materials, to verify the conformity of the material properties with the design assumptions.

Whereas the first two categories of tests are carried out on a daily routine basis, the third category tests are performed only periodically, e.g. once a year, or as required by the designer.

An example of the minimum frequency of these tests, as implemented at La Grande Phase I, is given Table VI.

The test results are communicated to the construction supervisors for immediate action and then processed using computer facilities to get average value, standard deviation, out of specification results and cumulative data, and to prepare spatial distribution charts for water content, fine and coarse fraction content, percentage density, borrow pit location, time of placement, etc.

FOUNDATION DESIGN

Dams founded on competent bedrock or dense glacial or fluvio-glacial deposits generally require a minimum embankment volume and conventional treatment to control foundation seepage and avoid internal erosion. When technically and economically feasible, the soft and compressible surficial deposits are therefore removed from the foundation limits of the embankments. At sites characterized by extensive deposits of loose or weak and compressible soil strata, the embankment design is adjusted with flat slopes and/or loading berms to control foundation stresses. Alternatively special mass densification or consolidation measures, such as, dynamic compaction, vibroflotation, preloading, sand drains, lime columns and the like, have to be adopted to improve the foundation bearing capacity.

Table VI

Embankment quality control - Minimum testing frequency - La Grande Phase I
(per cubic meter of material)

Material	Grain-Size Analysis	In-situ Density	Maximum Density	Water Content: Rapid Test	Water Content: Oven	Permeability: In-situ	Permeability: Laboratory	Specific Gravity and Hydrometer	Triaxial
Core	1/day or 1/5 000	1/day or 1/5 000	1/day or 1/5 000	1/1 000	1/5 000	2/season	1/200 000	1/50 000	1/500 000
Filter	1/day or 1/5 000	1/day or 1/5 000	1/day or 1/5 000			1/500 000			
Transi-tion	1/5 000	1/5 000							
Rockfill Shell	2/season	2/season							

Rock Foundation

The object of bedrock foundation treatment is to control seepage through bedrock, to avoid the presence of direct or preferential seepage along the interface between embankment and foundation, and to minimize, where required, the occurence of excessive arching effets, differential settlements and shear zones in the embankment. The extent of treatment, which may consist of minor to extensive reshaping, shotcreting and shallow and deep grouting, varies from project to project and even under different sections of the same dam.

The following standard rock treatment operations are carried out beneath the core and the upstream and downstream filter zones in all large dams:

a) excavation of all overburden and loose and fragmented rock;

b) reshaping of overhangs, steep faces inclined at more than 70° with the horizontal and other sharp irregularities by presplitting and light blasting or dental concreting to achieve a relatively continuous, smooth and uniform foundation surface;

c) excavation of shear and fault zones down to sound rock or to a depth at least three times the width of such a zone and backfilling with dental or drypack concrete;

d) surface cleaning with high pressure air and water jets to remove all mud, waste, dust etc.;

e) application of slush grout or pneumatically applied concrete where the rock surfaces are fractured, broken or characterized by open joints and depressions or where the rock is likely to deteriorate when exposed to air;

f) blanket and curtain grouting of the rock mass to variable depths or consolidation grouting of the fractured or sheared areas; and

g) low pressure contact grouting to fill voids between backfill concrete and rock surfaces.

Typical foundation treatments are shown schematically on Figure 12.

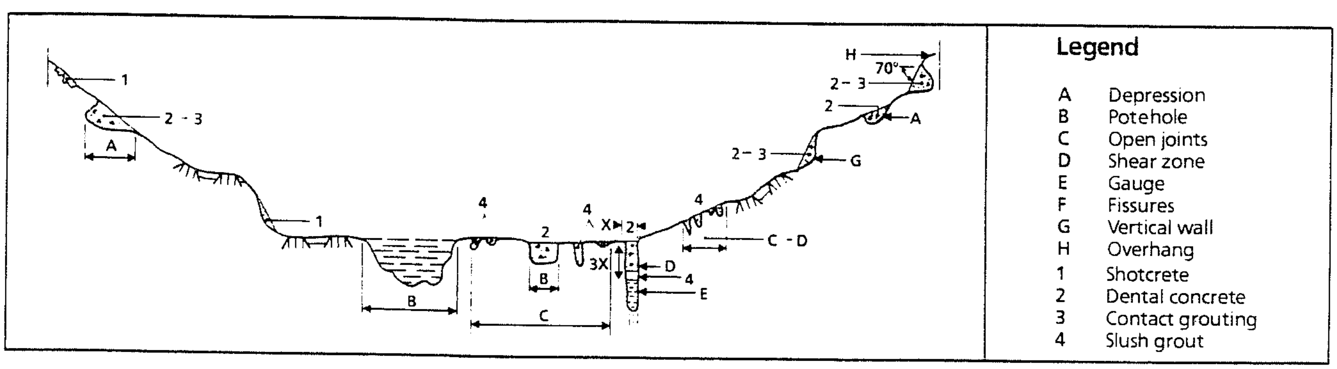


Figure 12 - Typical rock surface treatment

Treatment of the deep narrow gorge under the Lower Notch Dam in Ontario is shown on Figure 13. At the deep narrow part of the notch under Dyke QA 1 at LG 4, a concrete block has been used as foundation treatment (Figure 14).

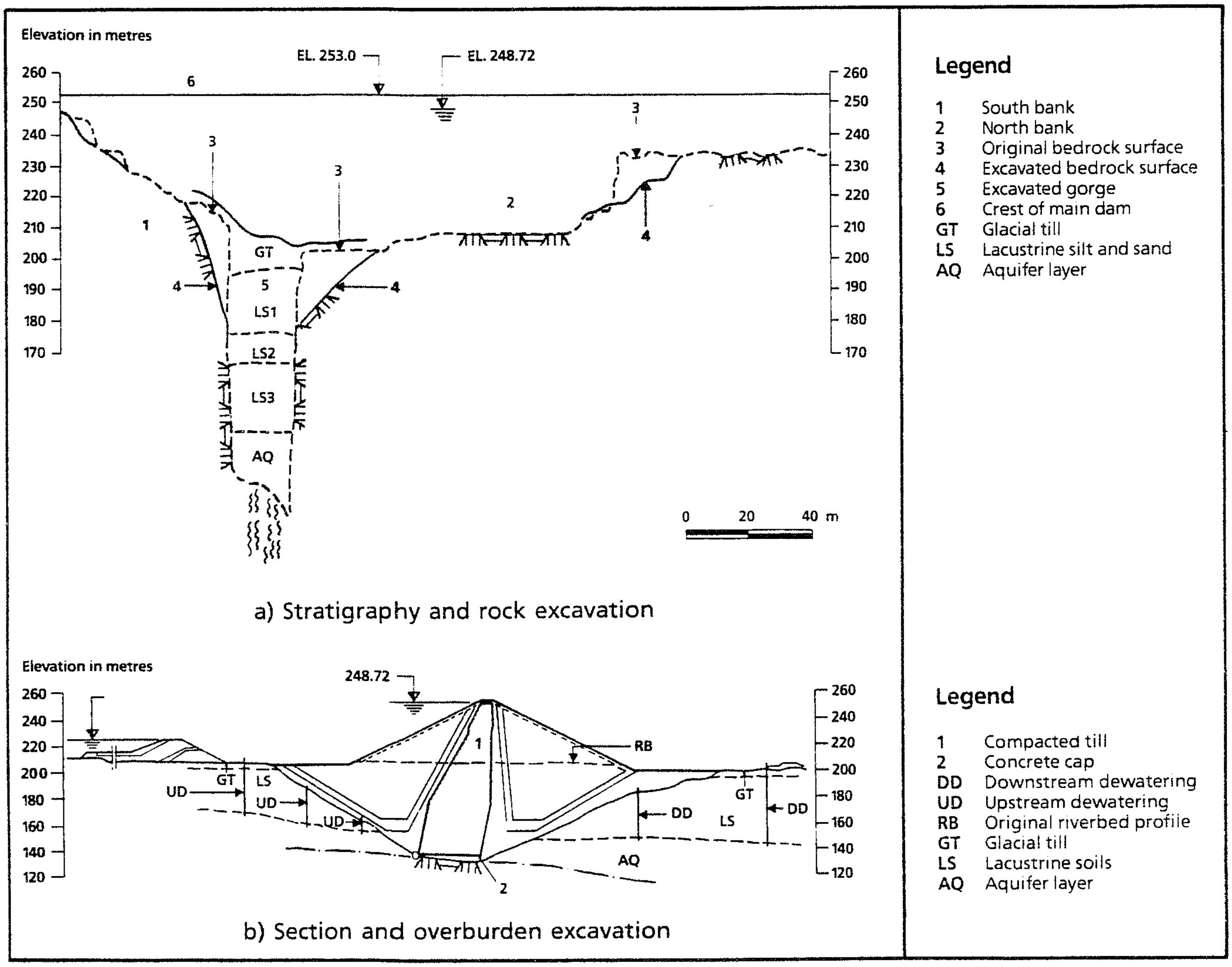


Figure 13 - Overburden excavation at Lower Notch Dam

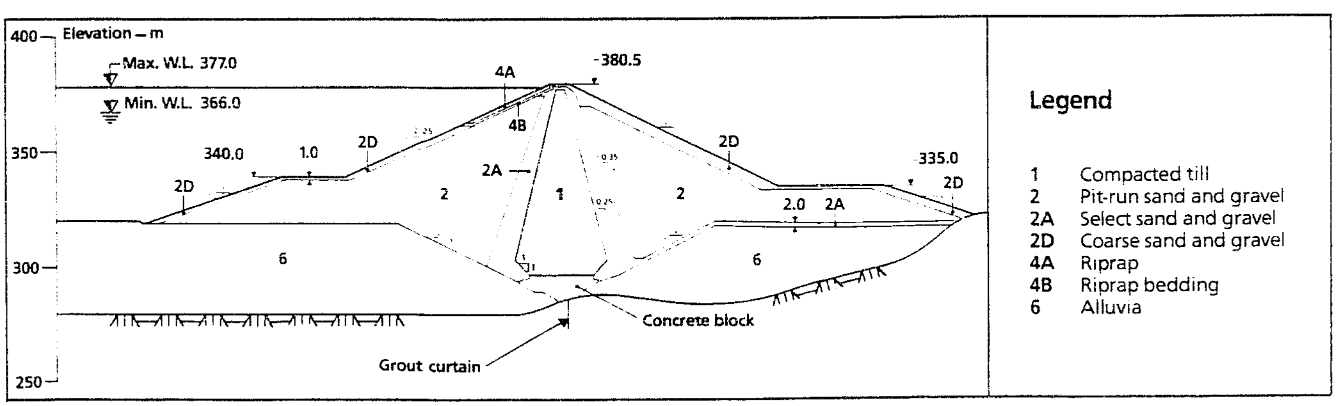


Figure 14 - QA 1 Dyke - Section in the deep narrow notch

The depth, orientation and spacing of the grout holes are designed initially with respect to the geological features of the bedrock established from exploratory borings and are modified during grouting operations as site specific conditions become known. An acceptable grout sealing of the rock was considered to have been achieved when absorption of the 5:1 water-cement grout in a 3 m long section of the hole was less than 15 litres in 10 minutes. To avoid hydraulic fracturing and lifting of rock, the maximum pressure was limited to 25 kPa per meter of depth of cover, with a maximum of 1500 kPa.

The grouting criteria, used at the La Grande Phase I sites are given in Table VII.

A summary of the quantities of rock foundation treatment works carried out at the different projects at La Grande Phase I Complex is presented in Table VIII.

As some post-construction seepage through bedrock in northern projects has been attributed to grouting of frozen rock, the bedrock temperatures were monitored with thermistors and no grouting was permitted at temperatures below 5°C.

Table VII

Rock grouting criteria - La Grande Phase 1

Grouting	Water Head (m)	No. of Rows	Spacing (m)*	Depth (m)*
Blanket	0-25	0	-	-
	25-60	2	6	8
	> 60	full core width	6**	8
Curtain	0-8	1	12	8
	8-25	1	6	8
	25-60	1	6	H/3
	> 60	1	6	H/3

* Intermediate and deeper holes added where needed
** Holes are split spaced

Table VIII

Rock foundation treatment - Summary of quantities - La Grande Phase 1

Site	Treated Surface (m^2)	Excavated Rock		Dental Concrete		Grouting		
		Total (m^3)	Per m^2 (m^3/m^2)	Total (m^3)	Per m^2 (m^3/m^2)	Holes (m)	Grout-take* (m^3)	(m^3/m)
LG 2	533 339	86 967	0.163	68 691	0.129	119 199	1 963	0.016
LG 3	328 429	186 101	0.567	51 174	0.156	72 781	959	0.013
LG 4	276 349	66 675	0.241	54 007	0.195	67 620	1 173	0.017
EOL	79 608	35 474	0.446	12 618	0.158	13 054	99	0.007
CANIA-PISCAU	615 512	122 638	0.2	55 871	0.090	80 137	701	0.008
TOTAL:	1 833 237	497 855	--	242 361	--	352 791	4 895	--

* Grout take measured in m^3 of dry cement

The zones of contact between the embankment and the concrete structures (such as diversion or intake conduits, concrete retaining walls or other permanent structures) are designed such that the post-construction stresses and deformations would tend to close the contact. The water tightness at embankment to concrete structure contacts is further improved by:

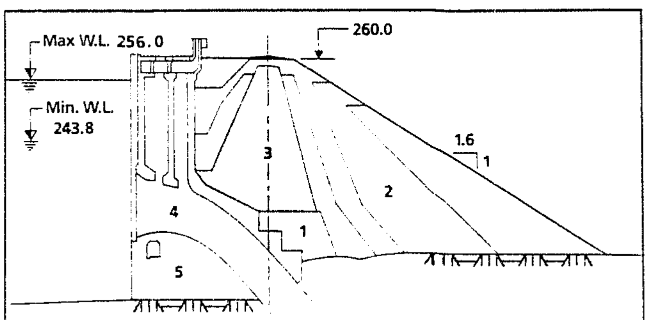
a) using selected core material of higher plasticity and water content,

b) flaring the core and filter zones at the abutment to lengthen the seepage path, and

c) limiting the concrete wall face batter to 70° and the downstream divergence to 85° with respect to the dam axis.

At the LG 3 site, where the water intake structure is located within the upstream shell of the embankment, a special arrangement was adopted for the core, filter and transition zones adjoining the concrete structure (Figure 15). Due to the slope of the downstream face of the structure containing the penstocks the core was widened and a concrete block was constructed under the core to provide a level foundation along the length of the intake. In conjunction with this, the filter and transition zones located downstream of the block were widened down to the bedrock by flaring and special grouting and drainage of the rock foundation was provided.

The reinforced concrete box of the diversion conduit at Dyke QA 8 was fully embedded in bedrock under the core and filter zones and was given a 70° batter under the shell zones where it was built on the bedrock surface. No cutoff projections or collars were provided to increase the seepage path.



Legend

1 Special concrete foundation block
2 Rock shell
3 Glacial till core
4 Penstock
5 Intake structure

Figure 15 - Intake structure connection with LG 3 main Dam

Glacial Till Foundation

The till deposits in Canada are generally dense to very dense and provide a competent foundation for embankment dams. These natural deposits contain layers, pockets and lenses of variable permeabilities and dimensions and demonstrate a "mass anisotropy" to seepage flow (Lefebvre et al, 1982, Paré et al, 1983, 1984b).

The treatment of the till foundation under the dam core generally consists of stripping, removal of disturbed and unsuitable materials, and recompaction of the acceptable foundation surface with slightly higher compaction energy than the one specified for the earth core of the dam. Surface wetting is employed when the approved surface is too dry for proper compaction. An exploratory core trench, generally excavated as part of the stripping operations, permits additional verification of the quality of the overburden foundation before embankment construction and may reveal conditions requiring local deepening of the core trench.

The inevitable small seepage occuring through the till foundation is generally handled by a filter zone located downstream of the core trench and connected to a horizontal blanket with a toe drain.

Since truly homogeneous and impervious deposits do not exist in nature, some undissipated foundation pore pressures transmitted to the toe should be anticipated and a toe drain is generally incorporated in the design in order to control exit pressures and minimize wet toe conditions. Based on analytical simulations of observed pore pressures and seepage behaviour of several cases, it was found that the "mass anisotropy" of till foundation permeabilities in the James Bay region ranges between 10 to 20. Furthermore it was observed that some 5 to 20% of the water head was transmitted to the till foundation under the toe (in conformity with the design assumption and analyses) and some seepage control measures were required.

Pervious Foundation

Treatment of the coarse grained pervious foundations is generally required to control the magnitude of seepage forces and ensure the stability of the embankment and its foundation against the risks of failure due to slope instability, internal erosion, piping and heaving. The treatment consists of installing a complete or a partial seepage control barrier or cutoff. The complete or positive cutoffs intended to reduce seepage to a negligeable amount are extended to the bedrock or the impervious stratum underlying the pervious formation. These cutoffs, in their generally recognized order of efficiency, are: core trench, concrete panel wall (often called slurry wall), slurry trench, grout curtain and sheetpile diaphragm.

The partially penetrating cutoffs, including the impervious upstream blanket, are designed to reduce the seepage pressures and quantities to an acceptable level based on economic

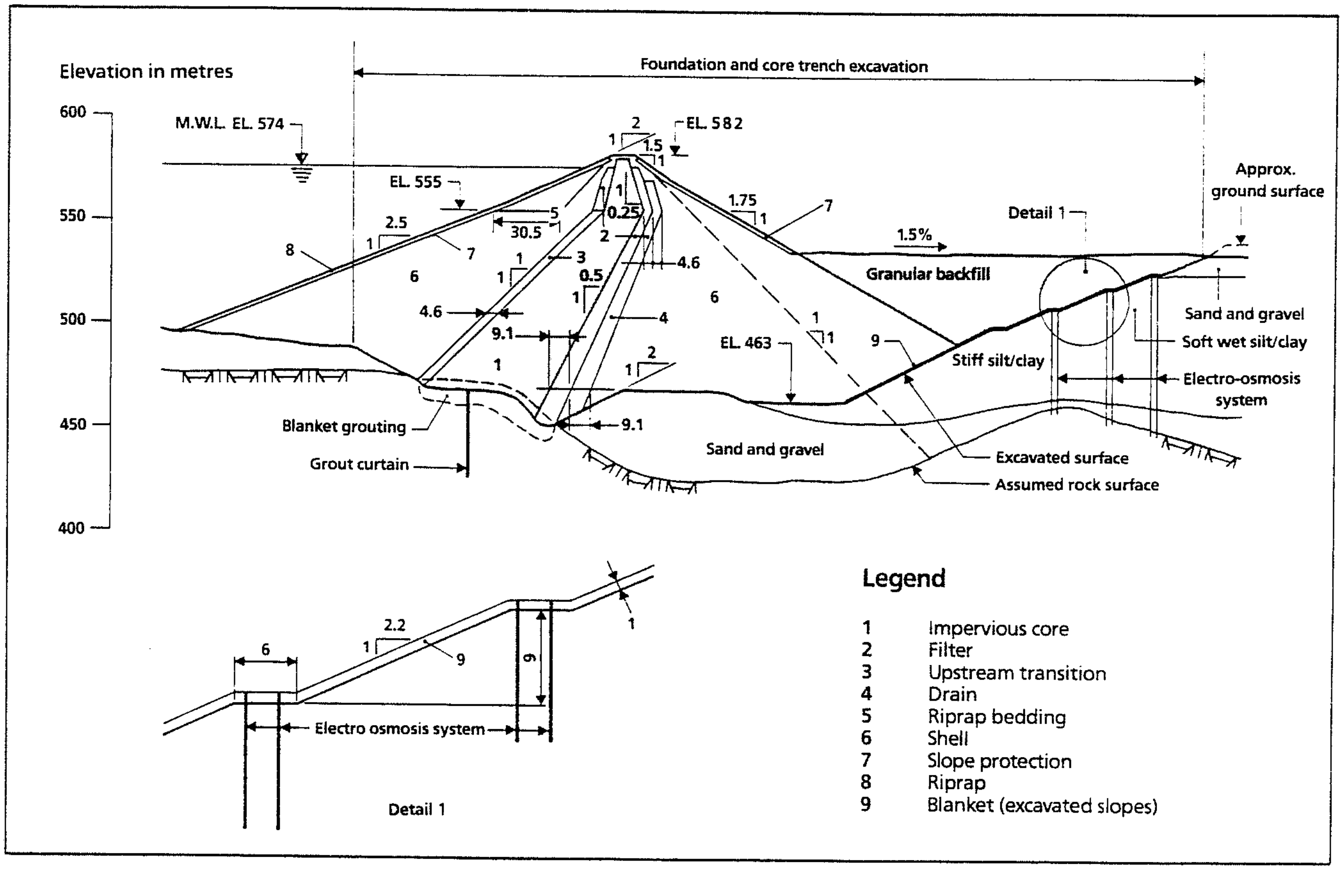


Figure 16 - Revelstoke Dam - Section and excavation details

and technical conditions and requirements. Partial cutoffs are always accompanied by some downstream drainage and protection features, such as a core drain, a horizontal downstream blanket, toe drains, relief wells and loading berms, to efficiently drain the seepage flow, control the exit pressures and improve toe stability.

The selection of a particular type of cutoff depends on the nature and depth of the pervious foundation, and the design philosophy (choice of complete cutoff and minimum surveillance versus partial cutoff and extended surveillance) and economic considerations including the value of the water losses. A brief discussion together with some well known examples of the various types of foundation treatments is given in the following paragraphs.

Where technically and economically feasible a complete barrier is always preferred. Among the various positive cutoffs, the core trench down to treated bedrock (employing the same impervious material and placement methods as the overlying core) provides the best type of seepage barrier. When required, it is complemented by a core drain and a downstream drainage system. Deep core trench excavations require overburden seepage control to safeguard the stability of the excavation slopes during construction. Dewatering methods most commonly employed are well points and deep wells (e.g. Lower Notch Dam and LG 2 Main Dam), pumping from sumps and electro-osmosis (e.g. Revelstoke Dam). The dewatering works are generally advanced in stages simultaneously with the progress of excavation. An interesting example of core trench cutoff and excavation slope stabilization works is shown for the Revelstoke Dam on Figure 16.

Among the major concrete wall cutoffs built in Canada, the Manicouagan 3 cutoff (Figure 10) consisting of two parallel concrete diaphragm walls with a 3 m spacing is of great interest. Each cast-in-place concrete wall consists of a series of interlocked piles in the central section of the 120 m deep burried canyon and of panels in the 30 m deep wing sections. The cutoff was overlain by an inspection gallery made of prefabricated concrete filled steel sections. A bentonite zone was constructed above the gallery to transfer some of the embankment loads away from the rigid walls. The Manicouagan 3 Dam has been well instrumented and it has been possible to monitor its behaviour and evaluate the effectiveness of the cut off wall system (Dascal, 1979). Other examples of concrete panel wall cutoff are: Arrow Lake cofferdam in B.C., Bighorn Dam in Alberta, and Dyke D 20, OA 11 Dam, LG 3 South Dykes and Manicouagan 5 upstream cofferdam in Quebec.

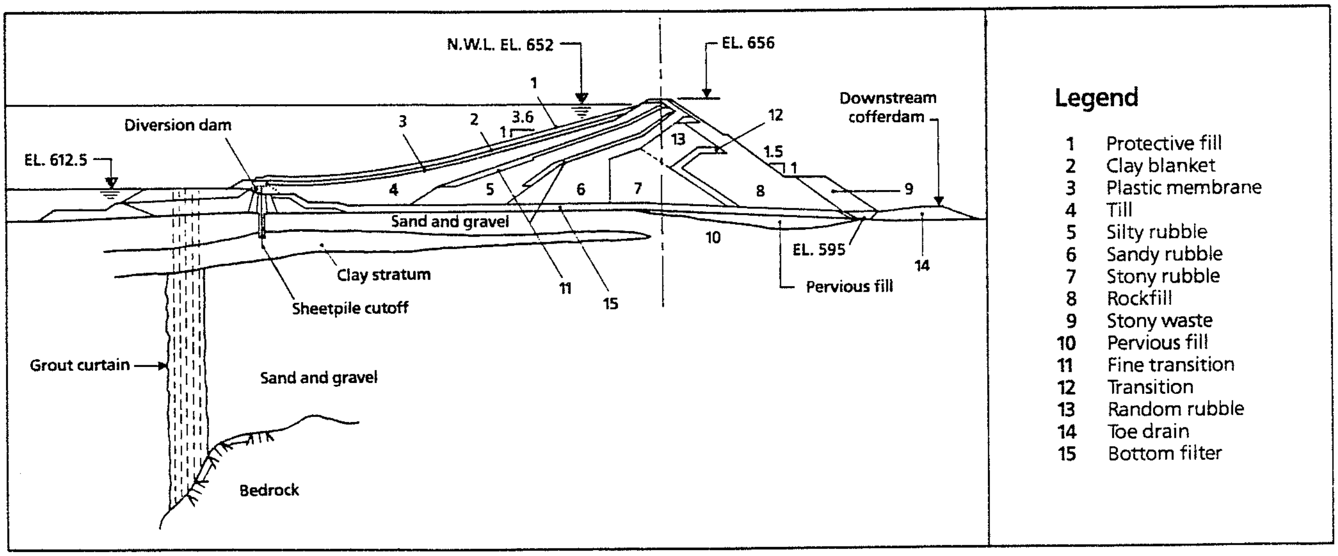


Figure 17 - Terzaghi Dam - Section and grout cutoff

The performance of slurry trench cutoffs (often referred to as Soil-Bentonite or SB cutoffs) has been "satisfactory" to "satisfactory after some additional post-reservoir filling stabilizing works". Seepage and high downstream pore pressures were noted locally along the toe of three dams with slurry trench cutoffs built at La Grande Phase I. The investigations revealed seepage to be related to the presence of very coarse or dense and permeable deposits in the niches and depressions of the highly irregular bedrock surfaces (Paré et al, 1982c, 1983 and 1985). Complementary works such as toe loading berms, grouting, relief well installations and additional instrumentation and surveillance were required during or after reservoir filling. The performance of the slurry trench cutoffs elsewhere has been considered satisfactory (Morgenstern, 1985).

The grout curtain cutoffs have been constructed at Terzaghi Dam (Figure 17) and Manicouagan 5 downstream cofferdam. The Terzaghi Dam (formerly Mission Dam) is founded on a deep buried valley filled with two pervious aquifers separated by a thick compressible clay stratum. The foundation cutoff consists of a grout curtain to a maximum depth of about 157 m through the lower aquifer and a sheetpile diaphragm through the upper aquifer (Terzaghi and Lacroix, 1964). The grout curtain under the Manicouagan 5 downstream cofferdam was built using a sleeve grout pipe device in which the design of grout mixes was controlled based on the results of monitoring the field operations and in-situ permeabilities. The efficiency of the grout curtains has been verified to be satisfactory at both sites.

Plastic cutoff walls consisting of self hardening grouts made from water-bentonite-cement mixtures with some retarding agents have been built in Europe in the past 15 years. Their construction is similar to that of the concrete panel diaphragm walls in many respects with the exception of their simplicity and continuity. The excavation operation is indeed continuous (rather than in alternate segments as for the panel wall) and the grout mixture, which acts initially as excavation stabilizing mud, hardens in place and creates a jointless cutoff. Such a cutoff was designed for some of the La Grande Phase I structures and proposed in the construction tender documents along with the alternative of the conventional slurry trench cutoff. The criteria used in the specifications were: sufficient deformability to adopt without cracking to the movements of the foundations, sufficient strength to withstand stress concentrations, sufficient imperviousness, and sufficient long-term stability against erosion. The contractor, because of the lack of North American precedent opted for the latter.

Compressible Foundation

The problems associated with the clay and peat foundations are related to their low shear strength and high compressibility. Excavating the compressible layer (fully or partly) and replacing it with suitable fill materials provides a positive but usually very expensive solution, unless the strata are of limited extent and thickness or the operations are justified by increased embankment height and reservoir capacity. The weak estuarine clay stratum and the overlying alluvium layer were removed by underwater excavation from the foundation limits of the Mactaquac Rockfill Dam in New Brunswick in order to locate the dam on suitable till foundation (Conlon and Ganong, 1966). Total excavation and replacement is generally adopted for peat deposits.

Improvement of the strength and compressibility characteristics of thick clay deposits by accelerating consolidation is a proven method of foundation treatment for high zoned embankment dams. This can be effectively achieved by means of carefully designed and installed vertical drains followed by a close monitoring of the field

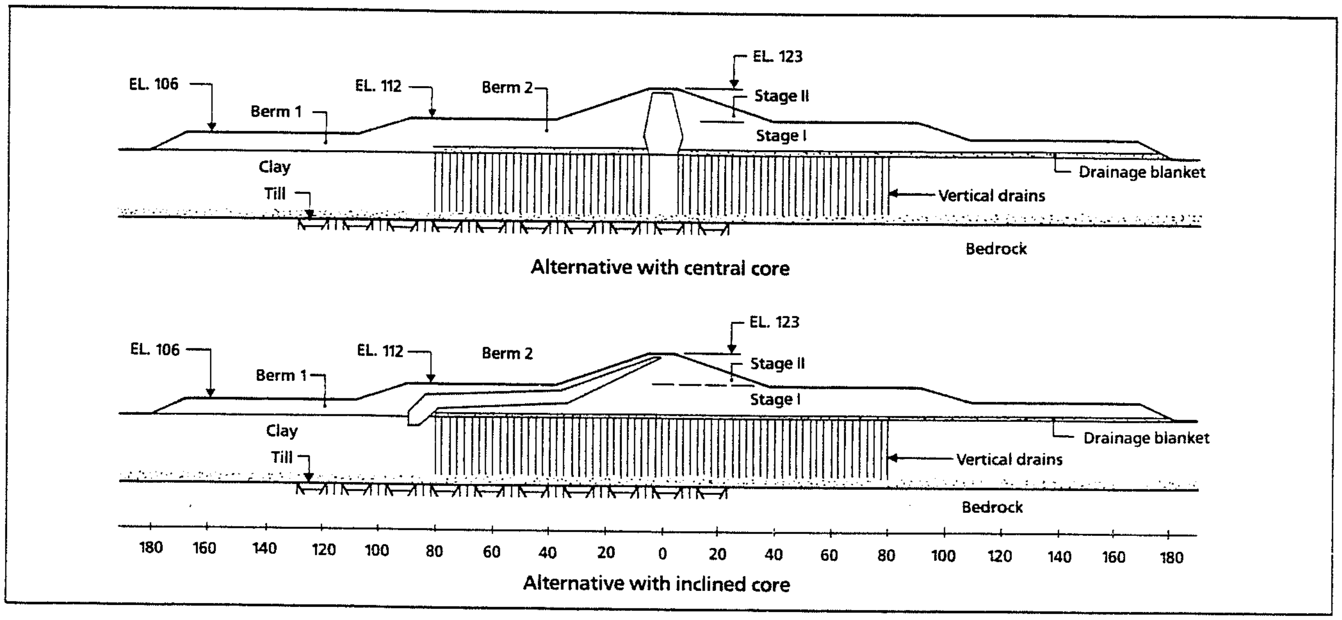


Figure 18 - B 6 Dam on soft sensitive clay (Proposed)

performance. Non-displacement sand drains and wick type drains are the two most popular of the vertical drains and the selection of a particular type is based on cost and performance. A test section might be required to establish the most efficient and cost-effective design of the consolidation system.

Other improvement methods, like lime column stabilization, fabric reinforcement, vacuum consolidation or groundwater lowering, may be used seperately or in conjunction with vertical drains to deal with especially difficult and local conditions.

The B-6 Dam at the proposed Nottaway-Broadback-Ruppert (NBR) hydroelectric development in northern Quebec is of special interest in so far as its height (23 m) and the nature of the underlying soft (40 kPa average undrained shear strength) and highly sensitive clay are concerned. A combination of vertical drains and multi-stage construction is regarded to be the most appropriate construction method (Figure 18). Following installation of the vertical drains, the first stage construction would consist of building the two 6 m and 12 m high berms. The second stage consisting of raising the dam to its final height would be implemented after a few months delay during winter shutdown when up to about 75% of consolidation would be completed. The 2 m thick peat layer would be removed from the central part of the dam (i.e. within the limits of 1:1 slopes) but would be left in place under the stabilizing berms. Beneath the other relatively low dykes of the NBR Project, it is planned to leave the peat in place under the full embankment width in consideration of the relative long seepage paths and low permeability of the compressed peat.

The Brazeau Development in Alberta consists of generally less than 9 m high canal dykes founded directly on compressible muskeg. The dykes have homogenous cross-sections (in clay and silty till) with a core trench cutoff through the muskeg overburden along the centreline of the dyke. The dykes built with exterior slopes of 2.75H:1V upstream and 2.25H:IV downstream required subsequent remedial works such as construction of downstream toe loading berms, rehabilitation of ripraps and grading of dyke crest (Wade et al, 1985).

Permafrost Foundation

The Kettle Generating Station Project on Nelson River in northern Manitoba is located in an area of discontinuous permafrost characterized by mean January, July and annual temperatures of -25°C, +10°C and -5°C respectively. The site conditions, project arrangement and design of dams and dykes have been described by Manitoba Hydro (1968) and Macpherson et al (1970). The 40 m high Main Dam and the 37 m high Saddle Dam consist of silty sand or sandy till enveloped by semi-pervious and pervious esker materials. Up to 16 m of permafrost affected silty clay and muskeg was removed to reach suitable till or bedrock foundation in the floor of the Saddle Dam. These excavations were carried out during winter months and the exposed frozen slopes were protected by a pervious fill zone.

In some locations, the dykes were founded on permafrost affected material with a high ice content. Thawing due to reservoir filling produced large amounts of free water and high pore pressures accompanied by reduced shear strength and subsequent large unevenly

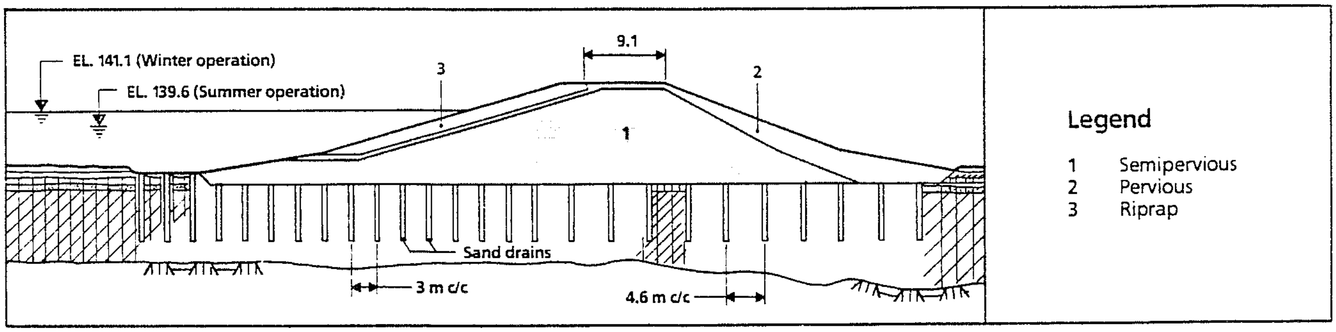


Figure 19 - Kettle Generating station - Dyke section on permafrost

distributed settlements. As shown on Figure 19, vertical sand drains were provided to reduce pore pressure build-up and accelerate consolidation. The dykes were constructed of sand to permit a great degree of settlement without failing and the dyke crests were designed to accomodate future settlement.

Based on experience gained at the Kettle Project, the dykes at the Long Spruce Project (McConnell, 1976) were also founded on permafrost and were designed with the following features:

- wide non-cohesive self-healing core as a defense against cracking (the dykes consisted of homogenous sandfill);
- flat side slopes in recognition of low foundation shear strength at the thaw front;
- drainage facilities, such as sand drains, to dissipate the thaw induced foundation pore pressure build-up;
- partial excavation of foundation materials such that the post construction settlements would not exceed 1.5 m and superelevating the embankment to minimize future maintenance associated with settlements; and
- instrumentation to monitor the performance of embankments and their foundations.

STABILITY OF RESERVOIR SLOPES

As a result of the development of a man-made reservoir, the perimeter ground slopes are subjected to unprecedented conditions of hydrological forces acting on the slope surface, changes in the groundwater regimes and sometimes, changes in the soil/rock strength and structure. A landslide in the reservoir could create waves that would breach or overtop the dam, interrupt operations, or cause extensive environmental damage around the reservoir or downstream of the dam. Depending on the location and the extent of instability, the results of slope movements can vary from inconsequential to catastrophic. Following some 500 landslides related to the initial filling in 1941-42 and the subsequent drawdown in the Grand Coulée Dam reservoir on Columbia River, the dam abutment failure at Malpasset in 1959 and the Vaiont reservoir landslide in Italy in 1963, there has been an increased interest in this important subject among geotechnical engineers.

The permeability and strength characteristics of the soils in the overburden slopes, the extent and orientation of the faults in rock slopes, the presence of old rupture surfaces or scars, the susceptibility of soil and rock to weaken on wetting, and the height and rate of reservoir level changes have an important bearing on whether the stability of the perimeter slopes would be affected favourably or adversely. Kenney (1986) has suggested that whereas the stability of the clay slopes would generally improve with increasing reservoir levels, the stability of the permeable slopes and especially slopes with old rupture surfaces could be minimum at a critical reservoir level located in the middle-third of the slope height. With the rising reservoir level, the stability of the highly pervious slopes first reduces due to diminishing weight and shear strength of the toe block until about mid-height and then increases as the slide mass gets submerged and its stability is increased again to the case of no submergence.

Detailed studies were undertaken to investigate the initial and post-reservoir filling stability of potential slide areas at the Mica Dam and Revelstoke Dam reservoirs in British Columbia (Moore and Imrie, 1982). For the Revelstoke reservoir, the risk of flooding the toe of the old 1.6 x $10^9 m^3$, 2 500 m long Downie slide (Gardiner et al, 1976) revealed the slide mass to be in a state of incipient failure and theoretical stability reduction to be 2 to 11%. The slide mass was stabilized by a system of drainage tunnels positioned to reduce underground water pressures within the zone of sliding.

Predicting the occurence of the first time landslides around the rim of a new reservoir is difficult and requires good knowledge of the regional geology and detailed explorations of soil and groundwater conditions, evaluation of the changed hydrological, environmental and geotechnical conditions,

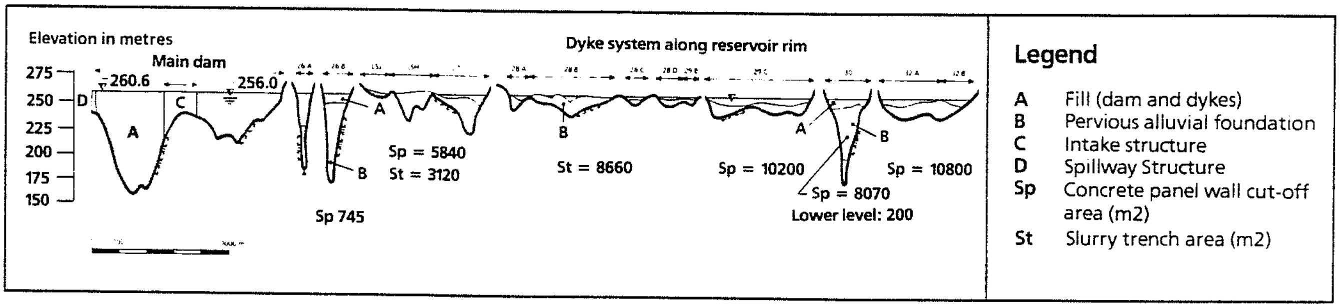


Figure 20 - LG 3 Reservoir rim - South dykes and cutoffs

and detailed analyses simulating the changes imposed in appropriate combinations. Such an investigation has been carried out for the La Grande river bank slopes located downstream of the LG 2 Main Dam (Rosenberg et al, 1984). As a result of the development of the LG 2 Project and greater demands for power in the winter months and reduced demands in the summer months, the river flow patterns have been changed from the natural spring floods and winter low levels to year-round regulated flows. This has resulted in new water levels and new periods and durations for water and ice action. Numerous shallow slips and a few deep-seated slides caused mostly by toe erosion under wind driven wave action have occured. All river bank instabilities have been mapped and documented, and a few of them have been investigated and instrumented. The resulting information will be used to evaluate the effects of the proposed LG 1 reservoir which will be located downstream of LG 2 and which will inundate these slopes. The typical hydraulic gradients in the clay slopes being downward (due to the presence of a more pervious till stratum beneath the clay), the effect of inundation for a certain level of the reservoir would be to reduce the margin of safety by counteracting the favourable effect of reservoir filling applicable to most clay slopes.

Investigation of the sensitive clay slopes of the Waba Dam reservoir at Arnprior in Ontario indicated a general improvement of the stability following the reservoir impoundment, although a few small shallow slope failures caused by wave action were observed.

Figure 20 shows the shapes of the perimeter valleys and the thick and highly pervious fluvio-glacial deposits underlying the low structures of the LG 3 South Dykes. During impoundment, excessive seepage and sinkhole, piping and slope instability activities occured downstream of the dykes as the reservoir approached the maximum level. To remedy the problem, the reservoir level was lowered by 2 m and extensive repair works, such as installation of deep drainage, construction of loading berms and panel wall or slurry trench cutoffs (Paré et al, 1985), were carried out. Since then the performance of these dykes under full reservoir condition has been considered satisfactory.

SEISMICITY

Effect of Seismic Forces on Dams

The embankment dams and their foundations are susceptible to the detrimental effects of the seismic forces resulting from the natural activities originating within the earth crust as well as the activities induced by reservoir filling and fluctuations. The recently revised seismic risk maps included in the National Building Code of Canada provide peak firm ground accelerations and velocities (PGAs and PGVs) corresponding to a change in risk level from 1/100 to 1/475 and reflect the fact that the seismic risks in Canada are higher than recognized 25 years ago. The Pacific coast with earthquakes on the Richter scale of up to M7 to M8 is one of the most active areas in Canada. Earthquakes of magnitude M6 which occured in 1985 and 1986, i.e. after construction of several large dams in British Columbia, had little if any effect on these dams (Ridley et al 1986).

Based on a detailed study of the known failures and damages to the embankment dams caused by earthquakes in the United States and Japan as well as the many cases where no failure or damages occured during earthquakes in the U.S.A., Japan, South America and Russia, Seed (1979) and Seed et al (1981) have made the following general conclusions regarding the performance of the rolled embankment dams during earthquakes:

1) virtually any well-built dam on a firm foundation can withstand moderate earthquake shaking (up to 0,2 g) without detrimental effects;

2) dams constructed of clay soils on clay or rock foundations have withstood extremely strong shaking, ranging from 0,35 to 0,8 g, from a magnitude M8 earthquake with no apparent damage;

3) two rockfill dams have withstood moderately strong shaking with no significant damage; rockfill kept dry by means of a concrete facing should experience only small deformations;

4) dams constructed of or founded on saturated cohesionless soils and subjected to strong shaking can suffer damage or failure due to pore pressure build-up and the corresponding loss of strength; and

5) the fact that a number of dams have failed in periods up to 24 h after an earthquake suggests that piping through cracks resulting from shaking may well have been responsible for the failure.

ICOLD (1983) has recommended the use of two earthquake loading conditions referred to as the Design Basis Earthquake (DBE) and the Maximum Design Earthquake (MDE) for evaluating the stability of the dams. The DBE with an annual probability of 1/475 is defined as a moderate earthquake which has a relatively high probability of occuring during the operating life of the structure. The MDE is described as the earthquake that would produce the maximum ground motions which the dam would be required to resist while maintaining its water retaining capability. The level of risk for the MDE depends on the hazard rating of the dam.

Induced Seismicity

Simpson (1975) has reported many cases of man-made water storages where changes in seismic activity have occured during reservoir impounding and operation. These induced earthquakes range from major earthquakes with magnitude greater than M6 to micro activities detectable only with high sensitivity instruments. It is likely that because of the lack of appropriate instrumentation the micro activities have passed undetected at a large number of reservoirs. Some of the experience acquired at the Hydro-Quebec and B.C. Hydro dams is summanized hereafter.

Manicouagan 3

An earthquake of magnitude M4 occured approximately 10 km upstream of the dam on October 23, 1975, during reservoir filling. Hydro-Quebec decided to stop the filling of the reservoir momentarily and to evacuate the site. When investigations revealed no anomaly, the reservoir filling was resumed at a slower rate and three portable seismographs were installed to better define the location and magnitude of the activities. After monitoring for about one year, the aforementioned seismic event was considered to be a typical case of induced seismicity (Leblanc and Anglin 1978).

La Grande Phase I

The complex is located in Seismic Zone 1, a relatively quiet zone with the ground motion corresponding to 0.03 g. Based on information obtained from the Earth Physics Branch in Ottawa, the intensity of the natural seismic events in the James Bay region has varied between M2 and M5.

At LG 2 where reservoir filling was monitored by an array of seismic instruments, over one hundred induced micro-earthquakes smaller than M1 were detected (Buchbinder et al, 1981). The monitoring of LG 3 reservoir filling revealed a number of small (up to M3) induced earthquakes. This significant activity has been related to a mapped fault and to the intersection of two dominant regional geological features. The induced activity started two months after impoundment commenced, when the water depth had reached 45 m and continued for the remaining 15 months of reservoir filling to a maximum depth of about 80 m. Subsequently after a lull of about 6 months when the water level was lowered by about 2m, a short burst of events, much more energetic than those during the filling, occured. Similar behaviour has been repeated several times during subsequent cycles of reservoir level fluctuations (Anglin and Buchbinder, 1983).

B.C. Hydro Dams

No published records are available relating to the induced seismicity for the dams in British Columbia. Following the natural earthquakes of 1985 and 1986 which were felt by the operators in the powerhouse at the W.A.C. Bennett and the Peace River Canyon dams, B.C. Hydro has initiated a programme of reviewing and analysing all the pertinent data and installing more sensitive seismographs with a recording range from 0.01 g to 1.0 g (Ridley et al, 1986).

Design to Resist Seismic Forces

Pseudo-static analyses have been used traditionally to evaluate the safety of embankment dams against sliding failure. These methods account for the earthquake forces by arbitrarily increasing the horizontal forces and do not account for the dynamic response or the pore pressure build-up.

B.C. Hydro has initiated a programme of reassessing the safety of some 50 dams with respect to the applicable DBE and MDE solicitations. The approach (Ridley et al, 1986) is to use pseudo-static analysis and conservative assumptions for initial indication of the seismic resistance available in an embankment-foundation system not subjet to significant dynamic response or pore pressure build-up due to earthquake shaking. Where this simple method indicates a potential stability problem, more rigorous analytical methods are used to verify the findings. Where embankment and foundation are vulnerable to excessive strength loss and pore pressure build-up, other methods such as dynamic finite element analyses have to be used. From the knowledge of the pore pressure build-up, together with the resulting soil deformations and the in-place strength characteristics, the factor of safety against failure both during and after the earthquake is evaluated. The structures which are found to have inadequate resistance to the design earthquakes are rehabilitated generally by adding confining/weighing rockfill masses to upstream and downstream slopes, improving the drainage system, constructing panel wall cutoffs, replacing weak fill material and densifying loose foundation zones, or are replaced by a new dam.

Proper assessment of the safety of an embankment dam involves significant practical experience and sound engineering judgement in addition to numerical analyses based on carefully selected parameters and properly simulated conditions. Safety of a dam can be ensured by carefully designed and constructed lines of defence. Seed (1979) has given a list of defensive measures to eliminate some of the potentially harmful effects of earthquakes on embankment dams. The list recommends the following provisions:

(1) ample freeboard for settlement, slumping or fault movements,

(2) wide transition zones of materials not vulnerable to cracking,

(3) chimney drains near the central portion of the embankment,

(4) ample drainage zones to allow for possible flow of water through cracks,

(5) a wide core of plastic materials not vulnerable to cracking,

(6) a well-graded upstream filter zone to serve as a crack-stopper,

(7) crest details which will prevent erosion in the event of overtopping,

(8) flaring the core at abutment contacts,

(9) core located to minimize the degree of saturation of the materials,

(10) prevention of slides into the reservoir.

DAM SAFETY AND SURVEILLANCE

Dam Safety

There have been about 100 major dam failures in the U.S.A. since 1930 and at least 21 incidents are considered to have occured to the intermediate and large dams in Canada over the past 70 years (Anderson, 1986). With the technological advances of the recent decades in the fields of geotechnical engineering, construction technology, instrumentation, communications and artificial intelligence, there has been an increase not only in the recognition of the need to ensure safety of dams but also in our ability to understand and rapidly analyse their conditions.

In Canada, the responsibility to ensure that dams are designed, constructed, operated and maintained according to recognized standards and procedures belongs to provincial government agencies. Before any new dam is constructed a certificate of environmental approval, based on the laws governing the river and stream alteration, and signed by the Ministry of Environment or the Ministry of Natural Resources, must be obtained by the owner. Anderson (1986) has listed the following areas of responsibilities for the provincial dam safety agencies:

- to keep an inventory of dam location and characteristics;
- to assure that hazard potential is assessed at regular intervals;
- to assure that design, construction, operation and surveillance are done properly and that the records are properly maintained;
- to provide an independent verification of the basic design assumptions; and
- to assure that emergency plans and procedures have been established where a high potential for loss of human life exists.

At the present time Alberta is the only Canadian province which has created the "Dam and Canal Safety Regulation Act". The other provinces are presently in the process of planning safety programmes and developping regulations and policies to ensure the safety of all existing and future impoundment works.

Legally, the owner is responsible for the safety of the dam at all stages and times. Provincial hydro and power authorities have in-house know-how and equipment to carry out design, construction, operation, surveillance, maintenance and repairs. Other owners do not always have adequate expertise or financial resources.

All aspects of the design and construction of a large dam project starting from the very conception of the idea are carried out by experienced specialists with assistance or major involvement, where required, of consulting firms and specialized contractors. These engineering works are generally carried out under the supervision and control of a review board or a design panel comprised of experts in the related fields. The mandate of these boards can be limited to a specific aspect of the project or be very broad and include reviewing all engineering, construction and scheduling activities.

In as much as quality assurance is an inherent part of any project, boards make important contributions which are in the interest of both the owner and the public. The involvement of acknowledged experts greatly enhances the quality of work by stimulating the specialists working on the project and their aggregated experience and judgement effectively minimizes the possibility of overlooking any important feature or aspect that could effect the safety of the dam (Matich 1986).

Surveillance and Instrumentation

The performance of a dam and consequently its surveillance and maintenance requirements depend on the quality of the design and construction. A comprehensive field quality control is the assurance of achieving a well built structure, and a comprehensive surveillance is the assurance of its safe operation and long-term security. Surveillance comprises of detailed visual inspection, reliable field instrumentation and careful interpretation of the information. Well defined chains of command and lines of communication between the field inspection crew and the decision makers in the office are important to ensure timely actions.

In visual examination, particular attention is paid to the changes which may occur in the drainage pattern (including appearance of wet or soft zones, colour and turbidity of seepage, and boils), the water levels, the exposed surfaces of the structures (appearance of fissures, sink holes and deformation) and the natural slopes close to the structure or on the periphery of the reservoir.

Some instrumentation is common on most large dams and it is generally concentrated in areas of critical or unusual conditions or in areas of special design feature. The objectives of these installations are:

1. to verify the soundness of a new design and to verify that the behaviour during and after construction is acceptable and in conformity with the design hypotheses, and

2. to obtain sufficient measurement data and assess the strengths and weaknesses of the design approach, construction methods, analytical techniques and the instrumentation itself.

The instrumentation generally consists of weirs, piezometers, inclinometers, settlement cells and profilers, surface monuments, total pressure cells and extensometers installed at strategic locations in the embankments, foundations, abutments and critical slopes. Dams being structures of long life span, the instruments selected for their monitoring should be well installed and possess high long-term reliability. The instruments should therefore be simple in design, rugged and stable, and easy to read, maintain and repair.

Among many interesting examples of instrumentation provided for special purposes, a few cases are given as follows:

W.A.C. Bennet Dam (Figure 5): Instrumented to measure pore pressures, deformation, stresses and seepages in fill and foundation characterized by shale beddings and a mylonite seam sloping in the downstream direction; special features included foundation displacement points in tunnels and scratch plate devices in mylonite seam (Taylor et al, 1985).

Duncan Dam: Instrumented to monitor pore pressures and movements in the three dimensions of the dam and soft sand-silt foundation; a new settlement gauge for large anticipated settlements was developed; settlements of up to 4,3 m due to foundation consolidation of the 36,6 m high dam were observed during construction; to account for the large differential settlement and to safeguard the dam against internal cracking, major modifications of embankment zoning were undertaken (Gordon and Duguid, 1970, and Gordon, 1971).

Gardiner Dam: Extensively instrumented to monitor pore pressures and deformations in order to evaluate the stability of the dam founded on highly over consolidated clay shales characterized by very weak shear zones; about 2 m of foundation settlement and over 2 m of lateral displacement occured during construction; based on behaviour during construction, major design modifications consisting of extensive slope flattening and toe berm loading were undertaken (Peters and Lamb, 1979).

Mica Dam: Instrumented extensively to monitor deformation and pore pressures in the core and downstream shell of the dam founded on a deep steep-sided valley and prone to excessive arching and cracking (Wesbster, 1970).

Manicouagan 3: Instrumented to monitor the seepage controlling efficiency of the core and the double concrete wall cutoff and to monitor the stress distribution around the bentonite cushion incorporated in the till core over an inspection gallery located on top of this rigid cutoff (Dascal, 1979).

Dyke D 20: Instrumented to monitor seepage, pore pressures and the effectiveness of the cutoffs (core trench, impervious blanket, concrete panel wall and slurry trench), provided in different sections of the long and deep pervious alluvium foundation (Figure 21) as well as to monitor effectiveness of relief wells (Paré et al, 1982c).

OA 11 Dam: Instrumented to monitor seepage and pore pressures across various foundation cutoffs installed in the glacial till and alluvial sand foundation with special emphasis on the analyses of the mass anisotropy of the deposits (Paré et al, 1983).

Rock Slopes at Revelstoke Damsite: Steep instability-prone rock slopes in micaceous gneisses interlayed with calcium silicates were instrumented to measure

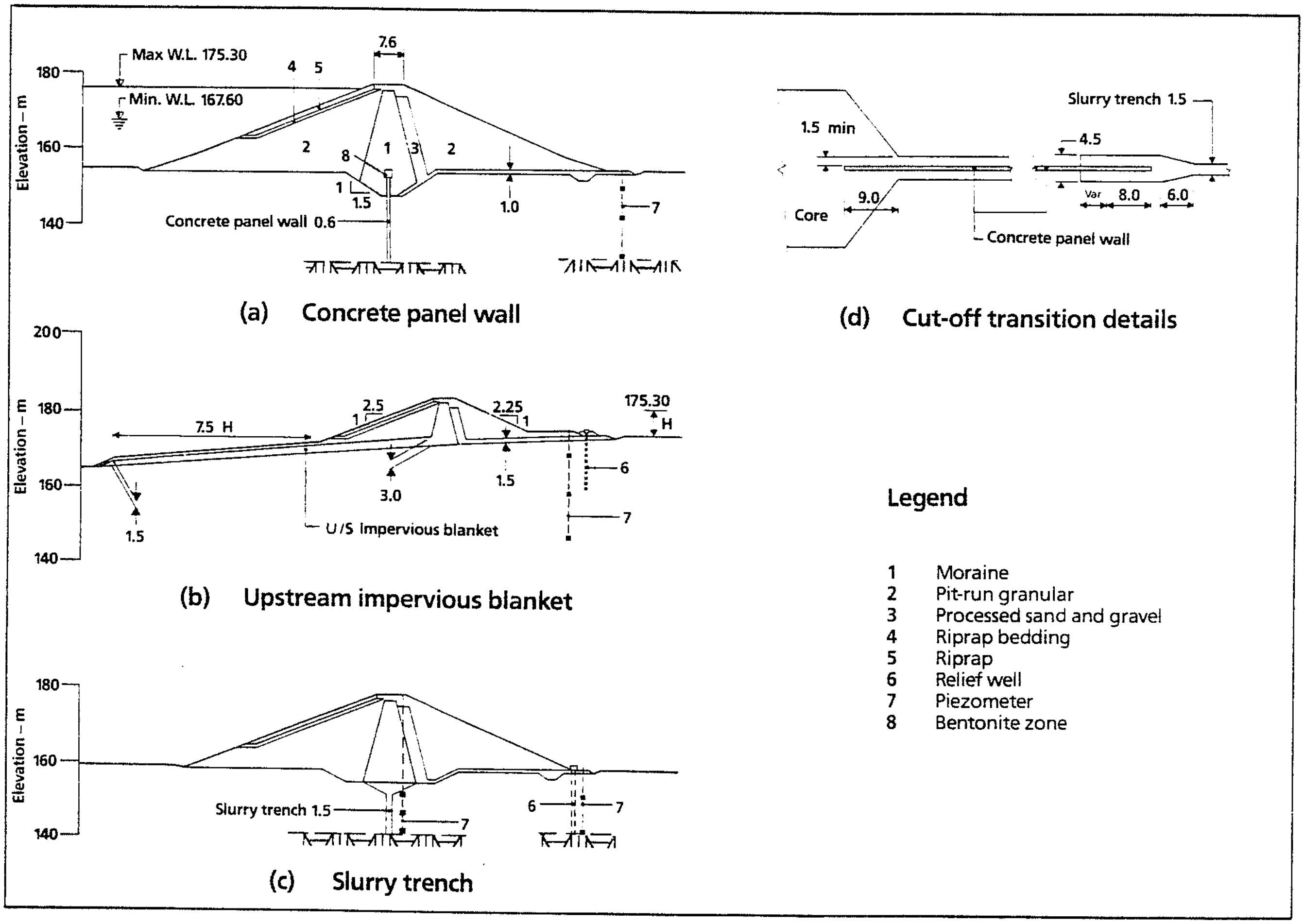


Figure 21 - D 20 Dyke - Sections and cutoff transition details

deformations and groundwater response (Moore and Imrie, 1982); slope designs and stabilization treatments were modified as a result of observations.

South Dykes at LG 3: Instrumentation was used to monitor the efficiency of the seepage control measures (panel wall, slurry trench with grouting in the lower part, core trench or deep drainage) installed under the low dykes located along the reservoir perimeter and founded on extensive alluvial deposits (Paré et al, 1985).

LG 4 Main Dam: Instrumented to monitor deformations, pore pressures and total stresses in the non-plastic till core at the zone of contact with the steep abutment and along the reverse curvature of the dam (Verma et al, 1985).

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Geotechnical Practice in Transportation - Highways

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ABSTRACT

Geotechnical requirements of a highway system are naturally governed to a large extent by the nature of the terrain transversed by the system. Geotechnical practices are developed to satisfy these requirements and may vary somewhat from region to region. In Ontario a tremendous diversity of subsurface conditions exists ranging from very soft organic and highly sensitive marine clays to hard or very dense glacial deposits, also many types of soft and hard bedrock. For highways, engineering works requiring a significant geotechnical input include bridges, tunnels, buildings, retaining walls, sewers and various earthworks and rockworks such as causeways, embankments and cuttings. This paper outlines the geotechnical practices followed in Ontario, including the field investigation and design phases in order to achieve a safe and economical design for these various engineering works. It is believed that these practices would apply elsewhere where similar subsurface conditions prevail.

RÉSUMÉ

Les exigences géotechniques d'un réseau routier dépendent naturellement, dans une large mesure, de la nature des terrains traversés par le réseau. Diverses pratiques, variables d'une région à l'autre, se sont développées pour satisfaire ces exigences. On trouve en Ontario une diversité incroyable de conditions de terrain, depuis les argiles marines très sensibles, molles et organiques, jusqu'aux dépôts glaciaires très denses ou très durs, avec plusieurs types de roches dures ou molles. Dans le cas des grandes routes, les travaux de génie qui nécessitent une composante géotechnique importante comprennent les ponts, les tunnels, les bâtiments, les murs de soutènement, les égouts, ainsi que de nombreux travaux en terre et enrochement comme les chaussées, les remblais et les coupes. Cet article expose, dans ses lignes générales, les pratiques géotechniques suivies en Ontario, y compris les méthodes d'investigation et les calculs afin d'obtenir une conception sécuritaire et économique de ces divers travaux de génie. On estime que ces pratiques pourraient s'appliquer ailleurs, pour des conditions de terrain semblables.

INTRODUCTION

This paper deals with geotechnical practice in highway design and construction based on thirty years of experience with the Ontario Ministry of Transportation and Communications by the authors. It is in no way intended to be used as a reference for geotechnical engineers except in a very general sense and the emphasis is on principles rather than detailed guidelines. The complete process of the geotechnical part of a civil engineering project which includes highways and various highway structures is described starting with the planning and execution of the subsurface investigation followed by the geotechnical analysis and design and finally the geotechnical requirements of the construction of the project. Included also is a description of the approach we have taken with regard to field testing of piles and soil and rock anchors. Within the Ontario Ministry of Transportation and Communications the Foundation Design Section has complete responsibility for all geotechnical design required for Ministry construction projects. This includes carrying out the necessary subsurface investigation and preparing reports containing the factual data obtained from the investigations together with recommendations pertaining to the design and construction of the project or projects in question. Preliminary and final design drawings must receive Foundation Design Section approval. All construction problems of a geotechnical nature are referred to the Foundation Design Section who will then determine the appropriate remedial action. Geotechnical consultants employed on Ministry projects work under the direction of the Foundation Design Section and are thus required to follow the same procedures and practices. It can be seen therefore that the geotechnical engineer working under such conditions has a large measure of control over the design and construction of civil engineering projects.

Unfortunately not all geotechnical engineers and particularly geotechnical consultants working for the private sector and some municipalities are able to operate in this manner. The majority will be involved only in the pre-contract engineering stage and this involvement usually terminates when they provide the foundation report.

SUBSURFACE INVESTIGATION

It is important to state the philosophy governing the scope and extent of investigations carried out to define subsurface conditions for civil engineering projects. Since it is first necessary to design the project and later to construct it, sufficient information must therefore be obtained to achieve both of these objectives. To achieve the most cost effective design it is necessary to examine all viable means of designing the foundations and earthworks and to recommend that all of these be considered by the designers. To enable the contractor to bid realistically on the project, it is necessary that he be provided with sufficient information for this purpose. It follows therefore, that the geotechnical engineer in determining the amount of subsurface investigation needed for a project must be experienced enough to be able to visualize the various methods by which the project can be designed and constructed. With this in mind, the planning and execution of subsurface investigations is conducted in the following way:

Planning the Investigation

All available documents which might be able to provide information on subsurface conditions at the site of the project are studied and assessed. These include existing subsurface data, air photos and geological reports. Information relative to the proposed works is studied carefully and a tentative program of field investigation is developed. In planning the program the geotechnical engineer sets out to define all soil or rock strata which will be influenced in a significant way by the construction of the project. For instance, an embankment constructed on cohesive soil requires investigation to a depth adequate to contain all possible deep seated failure surfaces. Similarly piles require that subsurface conditions are fully determined to depths below the tips of the piles when installed. During the actual fieldwork, modification to the planned program is usually necessary as the subsurface conditions are revealed.

Execution of Fieldwork

1. Boring Units

Fieldwork is carried out using equipment and operators hired from boring contractors. All field operations are supervised by a geotechnical engineer. (Geotechnical consultants hired by the Ministry must also provide a geotechnical engineer for field supervision). Boring equipment usually consists of continuous flight augers with 80 mm I.D. hollow stem or 100 mm diameter solid augers and conventional diamond drilling equipment adapted for soil sampling purposes. Various mounts are available for these drills depending on the nature of the terrain. For work on water the diamond drill is normally mounted on a raft or barge.

2. Sampling

Soil samples are obtained mainly by means of 50 mm O.D. standard split spoon samples or by 50 mm I.D. shelby tubes. Occasionally it may be necessary for soft sensitive soils to use piston type samplers to ensure sample recovery. For some projects block samples are obtained. Very occasionally 75 mm I.D. shelby tubes have been used. Samples of bedrock are obtained by coring.

3. Field Testing

Normal field tests are the standard penetration test, the dynamic cone penetration test and the field vane test. The first two are used to provide a measure of the relative density of non-cohesive soil and the vane to provide a measure of the undrained shear strength of cohesive soil.

4. Groundwater

Depending on the soil type and permeability and the presence of artesian or perched water levels, groundwater conditions are defined by means of piezometers or by direct observation in the open borings.

5. Classification of Soil and Rock

Field borelogs are prepared by the geotechnical engineer supervising the field investigation as the work proceeds. All boring and sampling operations are carefully recorded together with the results of all field tests and groundwater measurements. Soil is classified in accordance with the Unified Soil Classification System. Rock is described according to its type and Rock Quality Designation (RQD).

Laboratory Testing

Following completion of field work samples are subjected to a program of laboratory tests, soil properties are determined for classification and analyses purposes. These are as follows: Atterberg

Limits, Bulk Density, Water Content, Mechanical Analysis, Unconfined Undrained Shear Strength, Drained and Undrained Triaxial Shear Strength and Consolidation characteristics.

GEOTECHNICAL DESIGN

The goal of the geotechnical engineer in design is safety, economy and satisfactory performance. In the majority of highway projects the main concerns of the geotechnical designer are one or more of the following: slope stability, bearing capacity, settlement, deep foundations, dewatering and earth pressures. The Ministry's methods of analysis and design has been developed over the years and are based on practical experience with thousands of projects. These are summarized as follows:

Slope Stability

This is probably the most frequently occurring geotechnical problem in the Ontario highway system because of the presence at many locations of organic deposits lacustrine clays and marine clays. In such circumstances cut and fill slopes require careful design to ensure an adequate factor of safety. This is achieved by stability analyses in terms of total stresses for embankment fills and in terms of effective stresses for cuts. Acceptable stability is achieved by flattening the slopes or by the addition of berms or benches. Alternative methods of achieving stability such as removal or part removal of weak material are also recommended when considered to be economically feasible. In some cases stage construction may be recommended, however, this method is somewhat inconvenient since delays in fill placement are required to allow porewater pressures in the foundation soil to dissipate. Stability problems due to base failure usually do not occur where the foundation soil is non-cohesive. However, problems can occur within the slope itself particularly when the material consists of fine grained sands and silts and are subjected to seepage forces. This applies not only to cut slopes but also in some cases to embankment slopes when the fill material is fine sand or silt. In this case the accumulation of water in the granular soil below the pavement occurs when subsurface drainage is poor (or non-existent as is usually the case). Thus seepage forces are introduced to the soil at the slope surface and failure can result. The solution is to remove failed material, provide subsurface drainage and blanket the slope surface with coarser granular material. The same solution can be applied to natural or cut slopes composed of fine grained sands or silts.

Design parameters for analysis of slope stability are selected by the geotechnical engineer following a careful review of all field and laboratory test results, including groundwater conditions. In general, for stability analyses in terms of total stresses it has been found that the most reliable values of undrained shear strength are those determined by means of the Ministry's field vane.

Bearing Capacity

Bearing capacity of the foundation soil is required when the possibility of shallow foundations is being considered. About half of Ontario's highway structures are supported in this way. Parameters for design are based on field and laboratory test results, these being standard penetration test 'N' values for non-cohesive soil and field vane and unconfined and triaxial compression tests for cohesive soils. The level of groundwater affects bearing capacity in granular soils and must therefore be established accurately. Two values of bearing capacity must be computed. One, the ultimate capacity of the soil to support the structure and two, the load which results in the maximum tolerable settlement. (This settlement is usually assumed to be 25 mm). Bearing capacities of rock and unyielding soil (glacial till) are chosen arbitrarily since calculated values produce footing sizes much too small to be practical. From the ultimate bearing capacity computed, or arbitrarily selected the safe bearing capacity can be obtained by dividing by an appropriate factor of safety (2.5 - 3.0).

Settlements

In the province of Ontario settlements of highways and highway structures are of major concern, particularly in areas of organic deposits and lacustrine and marine clay deposits. Settlements of embankments built on these types of deposits up to 1.5 m are not unknown. The duty of the geotechnical engineer is to first estimate the probable value of the settlement, then determine what measures must be taken to alleviate the consequences of the settlements. To aid in settlement prediction, the Ministry installed settlement gauges under a large number of structure approach embankments which were constructed on cohesive deposits in the large expansion which took place in the sixties and seventies. Monitoring was carried out for several years and in this way settlement correction factors to be applied to calculated values were obtained for several types of foundation soil. Computations of settlements due to consolidation are based on laboratory consolidation tests of undisturbed samples obtained during the subsurface investigation. Methods of alleviating the consequences of large settlements or in fact preventing or reducing them include the following: lowering the grade, lightweight fill, advance fill construction, surcharging and removal of some or all of the compressible soil if economically feasible. Where a structure is supported on piled foundations and the approach embankment is built on highly compressible soil it is usual to provide structure approach slabs. Where it is determined that settlements of spread footings would be too large to be tolerated by the structure, deep foundations may have to be adopted.

Deep Foundations

Where the upper subsoil layers are too weak to provide adequate support for spread footings, it is necessary to obtain foundation support from a deeper level by means of piles or caissons. About half of all Ministry structures are founded in this way on so called deep foundations, and the great majority of these consist of driven piles being steel H and concrete filled steel tubes and timber piles. In a few cases precast concrete piles have also been driven. Cast in-situ concrete caisson piles have also been installed where it has been proven to be the most economical method. The design of piles can present difficulties due to the unreliability of theoretical methods therefore load tests are carried out whenever justified by economics. The Ministry's policy for pile test programs for many years has been to test all piles to failure and to document carefully all test results including defining subsurface conditions at the test site to depths well below the test pile tips. This has resulted in a large data bank of pile load and extraction tests carried out using different types of piles in different subsurface conditions. Tests to determine lateral capacities of installed piles have also been performed and documented. In addition the efficiency of the Pile Analyser manufactured by Pile Dynamics Incorporated has also been evaluated during the pile testing program. The benefit to the Ministry of the testing program has been enormous in that recommended geotechnical capacities of driven piles are now much greater than the maximum values used by structural designers some fifteen years ago. With regard to cast in place concrete caissons the testing program has resulted in a large increase in geotechnical design capacities by a factor of three compared to the values recommended for many years by the private sector. The Ministry's findings in this regard were very quickly adopted by Municipalities and the private sector. For most of the piling projects required for the highway program, it is possible to utilize the information from the data bank to make an accurate prediction of pile capacities where subsurface conditions are comparable to those at a test site.

Dewatering

This is a very important aspect of geotechnical engineering and it is the duty of the geotechnical engineer to recognize the possibility of problems in this regard. A problem occurs when excavations are carried out below the groundwater level and the inflow of water is sufficient to disturb the foundation soil or the stability of the sides or base at the excavation. It is necessary therefore to design a scheme to prevent this disturbance from occurring. Usually the contractor is required to design this scheme therefore the geotechnical engineer must provide all information regarding subsurface conditions including groundwater conditions required for this purpose. In addition he must outline viable methods by which dewatering can be carried out successfully. He must also draw attention to the consequences of improper methods of dewatering. Dewatering problems occur mainly in fine grained granular soils which are highly susceptible to conditions of unbalanced hydrostatic heads. Dewatering can usually be achieved by the use of eductor wells, well points or the construction of suitable cofferdams around the soil to be excavated. Dewatering problems usually to not occur in cohesive soil except where permeable strata may exist close to the base or sides of the excavation.

Earth Pressures

The geotechnical engineer must provide all information regarding subsurface conditions to determine the pressure exerted by earth or rock on structures permanent or temporary. Parameters required are angle of internal friction, bulk density and earth pressure coefficients for non-cohesive soil and undrained shear strength and bulk density for cohesive soil. In all cases, groundwater conditions must be defined. For permanent structures, backfill usually consists of granular material in accordance with specifications, also in some cases rock fill is used. For these cases the geotechnical engineer must estimate the necessary parameters.

CONSTRUCTION INVOLVEMENT

For contract purposes the geotechnical engineer provides certain documents to be included with the tender. He reviews and approves structural and highway design drawings (with regard to foundations) and during construction he provides guidance and in some instances supervision to construction staff for tasks such as pile installation, dewatering and stage construction. Where foundation related construction problems occur they are referred to the geotechnical engineer for remedial action. A more detailed description of the various aspects of the geotechnical engineer's involvement in construction is as follows:

Contract Documents

The geotechnical engineer provides for tendering and construction purposes, for each contract where applicable, a drawing showing profiles and cross sections which show the inferred subsurface stratigraphy and groundwater levels. He provides also a report which describes the subsurface conditions in detail and contains all borelog sheets and summaries of all field and laboratory tests. It should be noted that all of this factual data is guaranteed by the Ministry to be accurate. This is unique in North America and probably elsewhere. In addition to this guaranteed factual data provided with the contract documents the entire project files of the Foundation Office are also made available on request to all contractors bidding on the contract and also to the successful bidder at any time he wishes. Other contract documents prepared by the geotechnical engineer include drawings and reports detailing the results of pile load tests, soil and rock anchor tests and groundwater pumping tests. Special provisions relating to foundation construction such as caisson installation, pile driving, stage construction and dewatering are also prepared.

Document Review

During the structural and highway design process, liaison is continuously maintained with the geotechnical engineer on foundation and related matters. His approval is required at the preliminary and final stages for all foundation designs.

Construction Control

During the progress of construction contracts, the geotechnical engineer is available to provide guidance and in some cases supervision to construction staff. Standard forms for the recording of pile driving data must be completed and submitted for all construction projects to the Foundation Office. The same applies to acceptance and approval data for installed soil and rock anchors. Where certain construction operations such as stage construction, caisson installation, pile driving, foundation excavation and dewatering may require on the spot decisions to be made, the geotechnical engineer is frequently required to be present.

SUBSURFACE INVESTIGATIONS AND GEOTECHNICAL REQUIREMENTS FOR VARIOUS HIGHWAY PROJECTS

The following are the main types of projects which are investigated and reported on by the geotechnical engineer working for a highway authority. The main geotechnical processes and requirements are outlined.

Bridges and Retaining Walls

The geotechnical engineer is provided with a request for a foundation investigation at a proposed bridge site and is given all available relevant information, i.e. centreline, profile grade and tentative span arrangement. He now develops his boring program in the manner described above ensuring that it will be adequate to provide information for the geotechnical design of approach embankments or cuts, structure foundations (shallow or deep foundations - as alternatives if appropriate) associated retaining structures (wing walls etc.) sewers and subsurface drainage. It must also be adequate for the contractor to be able to tender for the work and to enable him to compare alternative methods of construction. On completion of fieldwork an appropriate laboratory testing program is initiated and directed by the engineer. All factual data is finally assembled and compiled in a form to be included in a foundation investigation and design report and is carefully evaluated after which the geotechnical design for the project is carried out. The final report is now prepared and includes all factual data and specific recommendations for the complete geotechnical design and construction of the structure, associated works and approaches.

Tunnels and Conduits

Subsurface investigations for tunnels are planned and executed in the same manner as described for bridges but with a much greater emphasis on defining groundwater conditions and the stability of interim conditions of construction as well as the final end of construction case. Again in his final report the geotechnical engineer must provide recommendations for viable alternative methods of design and construction of the work. One very important point to remember is to seal all borings when completed since their presence could affect construction operations such as use of compressed air and unwatering of excavations significantly. Depending on the importance and magnitude of the tunnel project, deep test pits and extensive pumping tests are also carried out to more precisely define the subsurface conditions. This applies whether the tunnel is in overburden or in bedrock. The particular problem of locked in stresses within bedrock strata must be fully addressed.

Embankments and Cuttings

Subsurface investigations are carried out by the geotechnical engineer for all approaches to structures whether they be cuts or fills and at various other locations where it is believed that stability (or settlement) problems may occur and the normal design and construction standards may not be satisfactory. The normal slopes designed by the highway engineer are 2 horizontal to 1 vertical for earth embankments and earth cuts, 1-1/4 horizontal to 1 vertical for rock fill embankments and 1 horizontal to 4 vertical for cuts in bedrock other than shale which is treated as earth for this purpose. These slopes have been derived after long experience of construction of highways in Ontario, however, it is necessary to specify suitable materials and methods of construction to ensure the desired stability. With regard to cuts in bedrock the bedrock surface only is established prior to design and problems which occur during construction or later are resolved at that time by the geotechnical engineer.

Where a subsurface investigation is carried out it is sufficiently detailed to provide all information necessary to design and construct the cut or fill in question. The final report provides the factual data and all detailed recommendations regarding settlement, stability, subsurface drainage, stage construction and any other special treatment required.

Acknowledgements

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Geotechnical Practice in Transportation - Railroads

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ABSTRACT

Geotechnical Practice in railroad transportation was well established by the time of Canadian Confederation. It played a key role in construction of the first trans-continental railroad in Canada, the rapid expansion of the trans-continental system and continues to play a key role today. The applications of Geotechnical Practice throughout this development period are documented here. Most attention is given to the nature of contemporary Geotechnical Practice. Specifically, the individuals who are involved and the nature of geotechnical applications across Canada.

RÉSUMÉ

La pratique géotechnique en transport ferroviaire était bien établie lorsque le Canada devint une confédération. Après avoir joué un rôle clé dans la construction de la première ligne transcontinentale au Canada, l'expansion rapide du réseau transcontinental, elle continue de jouer un rôle clé aujourd'hui. On décrit les applications de la géotechnique pendant cette période de développement. On se préoccupe surtout de la nature de la pratique géotechnique contemporaine, et plus précisément des individus impliqués et de la nature des applications géotechniques à travers le Canada.

INTRODUCTION

The significance of Geotechnical Engineering among the Engineering Professions in Canada and to Canada as a nation is nowhere better illustrated than railroad transportation. This paper begins with a brief review of the historical and contemporary role of railroads in Canada. The importance of Geotechnical Engineering is outlined. The unique geotechnical requirements of railroad construction and operation are summarized and the diverse geotechnical conditions imposed by Canadian physiography are considered. The paper ends with on overview of the current CP Rail Rogers Pass Grade Revision Project which embraces all aspects of Geotechnical Engineering Practice.

HISTORICAL AND CONTEMPORARY PERSPECTIVE

During early exploration and development, the colonies of British North America consisted of small settlements. These were situated in areas easily accessible to tidewater with a few located along major waterways in the interior. Short sections of trail, later converted to road and occasionally railroad comprised portage routes between navigable waterways. No other transportation routes were available at distance from rivers and lakes, and hence widespread settlement could not proceed.

The essential role of the railroad in fusing the colonies and providing an economic stimulus for development was clearly understood at the time of Confederation. Section 145 of the British North America Act opens with the passage: 'Inasmuch as the Provinces of Canada, Nova Scotia, and New Brunswick have joined in the Declaration that the construction of the Intercolonial Railway is essential to the Consolidation of the Union of British North America...' When the Act was proclaimed in Ottawa on July 1, 1867 it explicitly linked the construction of a major railroad to the creation of the Dominion of Canada. Within a few years, British Columbia also entered Confederation with a commitment from the Government of Canada '...to connect the seaboard of British Columbia with the railway system of Canada...'(Legget, 1973).

The nature of Canadian railroads changed dramatically following Confederation. Systems grew from one main line along the St. Lawrence Valley and isolated sections connecting navigable waterways to effective regional rail transportation networks. The Intercolonial Railroad was built to link the Maritime Provinces with Montreal and Toronto. Surveys for the Canadian Pacific Railroad (CP Rail) to the west coast were initiated and within eighteen years of Confederation a railroad system spanned Canada from coast to coast.

Soon two additional trans-continental railway systems were under construction as well as numerous branch lines. This contributed to a period of frantic growth over four decades, ending shortly after World War I as illustrated in Figure 1. One of the principal objectives of the railroad systems was to stimulate traffic by encouraging settlement and agricultural development. Irrigation and drainage schemes were constructed to encourage agricultural development in areas which would otherwise generate little or no traffic. Port facilities were constructed and serviced by rail lines. Rail lines were built to access coal deposits used to fuel steam locomotives and for domestic heating. The railroad systems linked the British North American colonies and made Canada the country we know today.

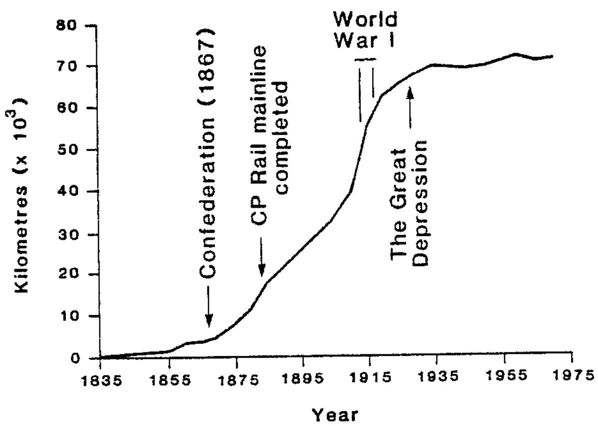


Figure 1: The growth of railroad lines in Canada.
- From Legget (1973) pp. 97.

Presently there are almost 70,000 km of railroad lines owned by more than twenty railroad companies. These traverse parts of all ten Provinces and both Territories. Two major companies, Canadian National Railway (CN Rail) and CP Rail operate the vast majority of the trackage. Their main systems, shown on Figure 2, provide a comprehensive east-west coverage through southern Canada. Significant northern extensions owned in some cases by the major companies provide access to iron ore reserves in northern Quebec, the tidewater port at Churchill, the Pine Point mine and Mackenzie River transportation system in the Northwest Territories and the agricultural, forest and mineral resources of northern British Columbia. Lead-zinc ore from the Yukon interior was transported by railroad to the tidewater port at Skagway, Alaska until an all-weather highway was completed recently. In addition, a route has been investigated from Hay River to Inuvik in the Northwest Territories with a branch continuing northwest through the Yukon towards Alaska.

Beginning in the late 1920's and continuing at the present time, Canadian railroads have faced intense competition from trucks for the transportation of high value merchandise. In response to the attractiveness of trucks, railroad companies now provide intermodal service in which highway trailers and containers are carried between major centres on rail cars. This combines the flexibility of highway carriers with the long haul efficiency of the railroad. Nevertheless, many branch lines serving smaller centres now have very light traffic due to loss of business to trucks.

Railroads are able to compete very well with any other means of land transport in moving

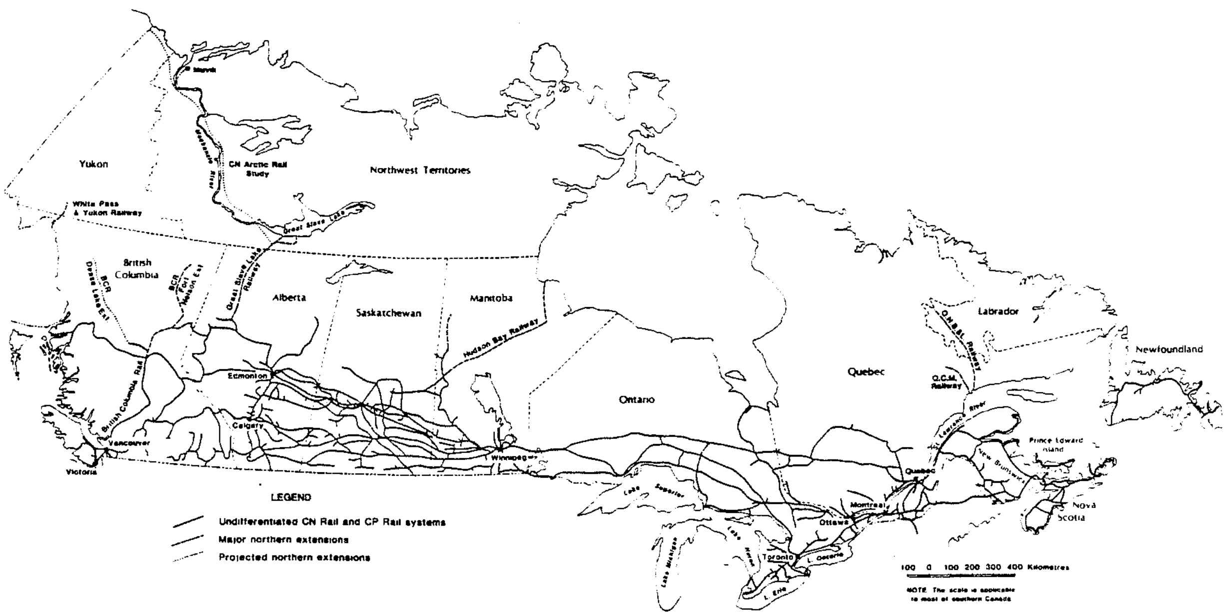


Figure 2: CN Rail and CP Rail Systems in Canada and major northern extensions of these systems. - From CN and CP Systems maps; QNS & L, Hudson Bay, Pine Point, and White Pass and Yukon Railroads added.

bulk commodities, especially when the bulk commodities can be moved in unit trains from a single point of origin to a single destination. Much of this type of traffic originates in western Canada and is destined for tidewater export terminals. Canadian geography dictates that these bulk commodities must be moved longer distances to seaport destinations than similar commodities in other countries. Hence, the cost efficiency of railroad transportation is a key factor in keeping Canadian bulk commodities competitive in the international marketplace.

THE IMPORTANCE OF GEOTECHNICAL ENGINEERING

Geotechnical Engineering emerged as a discrete discipline in the railroad industry in the late 1950s and early 1960s, but notable Geotechnical Engineering practices by Canadian railroad engineers span the period since construction of the first trans-continental railroad.

Many state-of-the-art procedures were used for the Intercolonial system. These include the use of soil penetration tests in foundation investigation and design (Peckover and Legget, 1973) and the use of preloading to reduce long-term settlement of bridge foundations (Legget and Peckover, 1973). Notable accomplishments from the original CP Rail system include construction of trestles of unprecedented length on the Prairies and height in British Columbia, and records for length of grade constructed per day using teams of horses for excavation. All of these are just a few of the enviable and impressive legacies to the Geotechnical Engineers of this design-build construction period.

Geotechnical Engineering practices continued to play a key role during the period of rapid growth between 1885 and 1925 with innovative developments in design and construction of main line, tunnels and even irrigation structures. One of the most notable was construction of the Grand Trunk Railroad tunnel through soft sediments beneath the St. Clair River at Sarnia in 1889 and 1890.

Geotechnical Engineering continues to play a key role today. Timely, efficient and reliable rail transportation is essential for railroads to maintain their current share of truck competitive traffic and to enable bulk commodities to compete in export markets. Compared to railroads in most other parts of the world, the North American systems, particularly those in Canada, handle heavier axle loads in longer trains and at higher speeds. When these conditions are combined with the long distances travelled, the severe Canadian climate and the constant pressure to maintain optimum cost efficiency to avoid loss of domestic or international business, it is apparent that geotechnical standards must be high.

GEOTECHNICAL DESIGN ASPECTS

Loads imposed by railroad traffic are distributed longitudinally by rails and transversely by ties which are embedded in ballast for stability. The ballast rests on a layer of sub-ballast which, in turn, rests on the subgrade. Stresses are much higher than in highway design because of the significantly greater wheel loads, and the concentrated loading imposed by train traction, braking and track curvature.

The optimum location for a railroad is the route offering the lowest possible gradients. The ruling gradient for modern heavy-haul railways is usually limited to a maximum of 1% ascending and this must be reduced to compensate for increased wheel friction in curves and air friction in tunnels. Even lower gradients are desirable since they permit more efficient operation by reducing the power required to haul a train of any given weight. Lower ascending gradients also permit longer heavier trains to operate over a single track railway providing greater railroad capacity without costly double track.

Horizontal curvature on new railroads is generally limited to 6^{0} (300 m radius) to control rail wear. Flatter curves permit higher speeds.

GEOTECHNICAL PRACTICE

The two major railroad companies operate with less than ten full-time Geotechnical Engineers. These few individuals concentrate on geotechnical problems associated with maintenance-of-way, structures and new construction at locations throughout Canada. Geotechnical Consultants are retained as required to assist with short-term manpower-intensive projects and to provide specialist advice.

These individuals are concerned with the availability and nature of borrow materials, a wide variety of engineered structures and slope stability like practitioners with any other linear facility. But railroads are also unique, in that design aspects limit the flexibility to avoid adverse geotechnical situations imposed by the geological continuum. Engineering geology plays an important role in providing an understanding of large complex geotechnical problems. Geotechnical practitioners with railroads must have sound knowledge of engineering geology because of the very significant cost implications of geological conditions on design, construction and operation.

The nature of the geological continuum is infinitely variable across Canada. Nevertheless, current Geotechnical Practice in rail transportation can be broadly categorized according to the physiographic regions shown in Figure 3 and to the nature of railroad system development; whether it is mature or there is ongoing construction of new line.

The Mature System

Most railway grade was constructed during the four decades of rapid expansion from 1885 to 1925 (Figure 1). As a result, the system in Canada is mature. The majority of geotechnical work is related to local settlement, deflections of ballast, sub-ballast or subgrade, or drainage problems, and these are attended to on a routine basis often without specific geotechnical engineering input. Problems of this nature are particularly common in the Canadian Shield (Figure 3) where there are hundreds of transitions between rock and soft organic soil

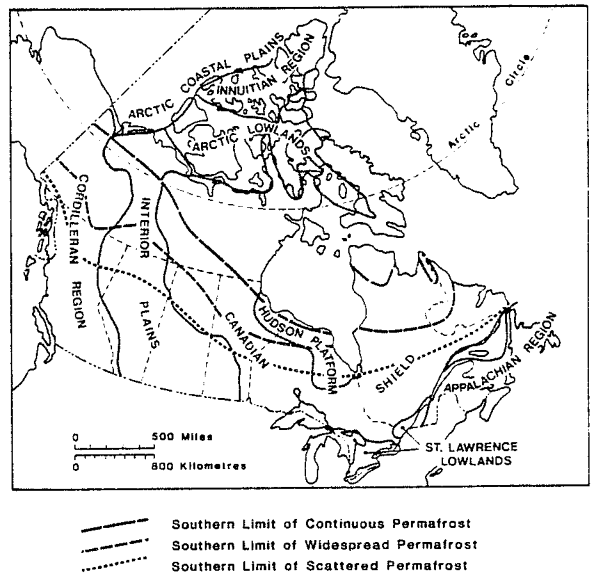


Figure 3: Physiographic and permafrost regions of Canada.
- From Douglas, pp.12 and Brown's Permafrost Map of Canada.

subgrade; and the St. Lawrence Lowlands and coastal portions of the Appalachian Region where much of the grade traverses soft and commonly sensitive marine clays and soft lacustrine clays.

Ballast degrades as a result of abrasion caused by cyclic loading (Klassen, et.al. 1987; Collins and Fox, 1985), and in some cases fouling with fines which are generated externally. Routine maintenance requires significant quantities of high-quality ballast. Suitable sources of ballast for main lines are available in all regions except the Interior Plains where quantities of suitable ballast are limited. Ballast for this region is also produced in the Canadian Shield and the Cordillera, resulting in haul distances of well over 800 km. Geotechnical Practitioners play a central role in identifying possible sources of acceptable ballast, in evaluating its properties and in developing quarry operations.

The Interior Plains region is typically an area of low relief with extensive tracts of stable high-speed grade. Although major river valleys in this region are relatively few, most are significant obstacles because they are deep and wide, having originated as glacial meltwater channels. Valley slopes are formed in soft flat-lying mudrocks and sandstones or glacial clays, all of which are commonly unstable. Highways, pipelines and transmission lines can be located to traverse these slopes quickly at steep gradients and at relatively stable locations while railroads must snake along unstable slopes for many kilometres. The CN Rail crossing of the Peace River Valley in Figure 4 is a good illustration of this. The section of line in the valley includes 22 km of grade traversing many tens of active landslides. Alberta Highway No. 2 meanwhile has only 11 km and a major Alberta Power transmission line has only 7 km in the valley. Where it is feasible, railroads use high trestles such as the CP Rail trestle at Lethbridge, Alberta (Figure 5) to cross these meltwater channels with minimal exposure to unstable slopes.

Geotechnical problems associated with slope instability typically involve slow ongoing settlement of the grade which requires regular track lifting or shifting of the track towards the hill. Occasionally the grade must be supported with simple tied-back retaining structures or placement of a stabilizing berm Earthworks must be undertaken with great care and due attention to the ever present possibility of initiating additional slope movement. Rock slope instability in the Cordilleran Region, and to a much lesser extent in the Canadian Shield and the Appalachian Region, is also a significant geotechnical problem. Frequent inspection of susceptible areas and regular programs of scaling and stabilization are the most common ways of dealing with this type of instability. All of these problems are dealt with by Geotechnical Engineers.

Major unexpected problems can occur in any area despite the mature nature of the railroad. Landslides such as illustrated in Figure 6 can occur anywhere during periods of peak runoff, during subsidences of floods, during drawdown of adjacent lakes or reservoirs, after changes in conditions of loading or through spontaneous liquifaction of sensitive marine clays (Smith and Peck, 1955). Wash outs, debris torrents and snow avalanches

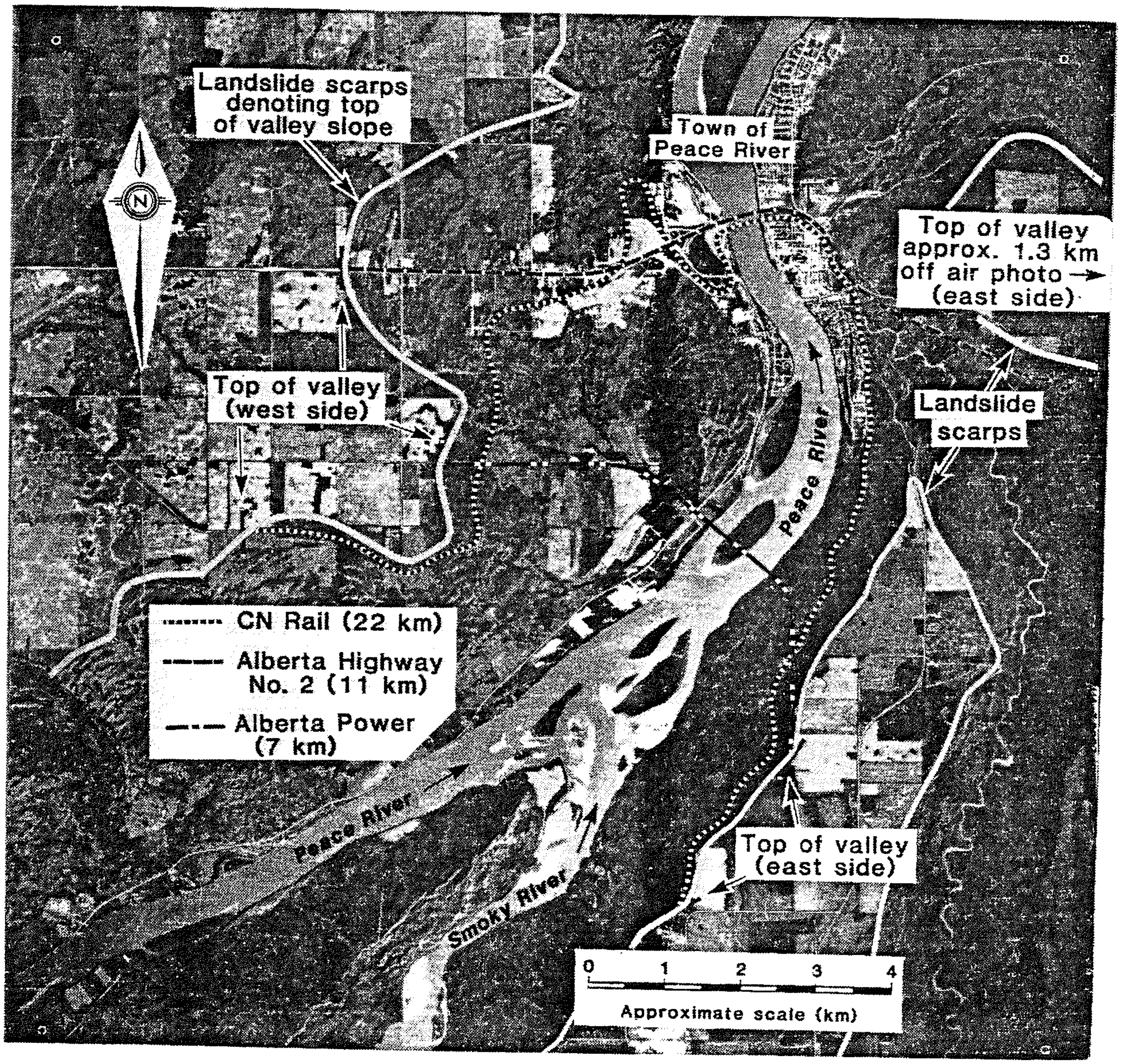


Figure 4: The CN Rail corssing of the Peace River Valley at Peach River, Alberta.
- From Alberta Energy and Natural Resources air photo AS 3057 34 (Scale 1:60,000) with annotations.

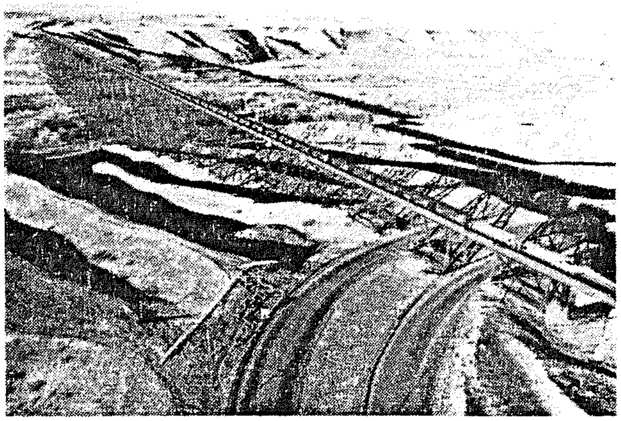
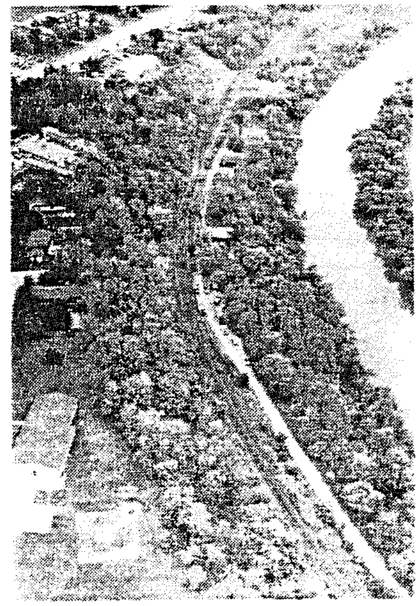


Figure 5: The CN Rail trestle crossing of Oldman River at Lethbridge, Alberta.
- Photo courtesy of CP Rail.



are other problems regularly encountered. Regardless of the nature of the problem, each demands immediate action to restore rail traffic while also giving due regard for the safety of persons and property in adjacent areas. Geotechnical Practitioners involved with these emergencies must quickly determine the nature of the problem and implement mitigative measures which enable rail traffic to resume with an adequate degree of safety. In emergencies, evaluation of the problem and development of the solution must often be done without complete topographic information or subsurface investigations. This underscores the need for Geotechnical Practitioners to have optimum regard for geological conditions.

Many northern extensions traverse parts of the discontinuous permafrost region of Canada shown in Figure 3. Some of these lines were built before the geotechnical complexities of ice-rich permafrost were fully appreciated. On the Hudson Bay Railway, which probably crosses more permafrost terrain than any other railway in Canada, railroad engineers in the 1920's anticipated the significant potential for problems associated with thaw-settlement and attempted to insulate and maintain permafrost (Charles, 1959). Nevertheless, serious thaw-settlement problems continue to this day in some locations. CN Rail has used state-of-the-art geotechnical designs to maintain the ground thermal regime in the frozen state on several sections of the Hudson Bay Railway (Haley, et.al., 1983).

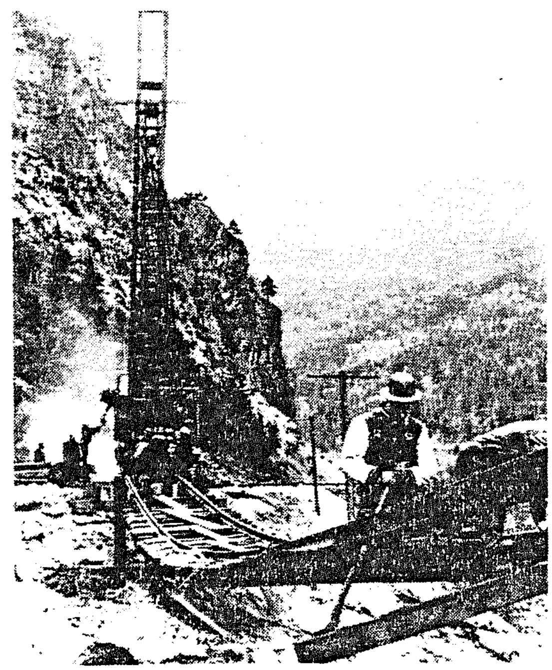


Figure 6: Remedial work on a landslide along the CN Rail line in the Canadian Cordillera. The landslide is to be bridged as part of remedial work. Within a few hours of shut down the pile driver in the background was on-site and operating. The line was re-opened with a timber bridge structure within 2 to 3 days.
- Photo courtesy of CN Rail.

Construction of New Rail Facilities

The rapid growth of a network of high-speed, high volume highways has required construction of numerous structures throughout Canada to safely separate railway and highway traffic. This has resulted in challenging work for Geotechnical Practitioners over the last three decades. These projects and pressure from agencies for major developments near the railroads has necessitated innovative geotechnical designs and construction techniques for bridges, retaining structures, tunnels and slopes (GEOSPEC, 1987).

The recent trend toward intermodal capability has required construction of rail to road transfer facilities in the major metropolitan centres. A unique geotechnical requirement in these areas is the need for pavements to handle unusually heavy wheel loads from transfer equipment which are commonly 400 KN (Peckover and Wong, 1971).

Major commodities moved in western Canada are grain, lumber, coal, potash and sulphur. As a result of growth of traffic in all of these commodities new port facilities have been constructed, notably the deep-water coal port at Robert's Bank south of Vancouver and the Ridley Island facilities for grain and coal at Prince Rupert. These facilities have presented new geotechnical challenges in offshore engineering and in situ testing.

Both major railways have found it necessary to increase the capacity of their main lines leading to west coast ports. CN Rail is in the enviable position of having the lowest ruling gradients (0.5% westbound and 0.7% eastbound) and lowest peak elevation (1,133 m) for any railway crossing of the Cordillera Region in western North America. It has increased capacity by installing an improved system of signals, by construction of longer regularly spaced sidings and by construction of some sections of double track, notably one section of 166 km in the Yellowhead Pass through which all CN Rail traffic for both Vancouver and Prince Rupert must travel.

The CP Rail profile in western Canada is compared to CN Rail in Figure 7. It can be see that the peak elevation and, as a result, the gradients significantly exceed those of CN Rail. CP is increasing capacity by reducing major bottlenecks with controlling gradients of up to 2.6%. Three major grade revision projects totalling 34 km in length were recently completed at Notch Hill, Clanwilliam and Stephen (Figure 7) to eliminate adverse curvature and reduce the gradient from between 1.6% and 1.9% to 1.0%. This work was completed in difficult mountainous terrain and involved extensive earthworks, including the use of electro-osmosis to improve engineering properties of organic silts prior to excavation of a deep cut (Nishizaki, 1982).

With these three projects completed, the most restrictive capacity restraint on the CP Rail main line between Calgary and Vancouver became the ruling gradient of 2.2% on the east approach to Rogers Pass (Figure 7). The Rogers Pass Grade Revision, which is nearing completion at this time represents the biggest undertaking by CP Rail since construction of the original main line. It involves construction of more than 33 km of new grade including the longest rail tunnel in North America. The new section with a reduced gradient of 1% compensated will be used for heavy westbound traffic. Lighter, eastbound traffic will continue to use the existing CP Rail line. Geotechnical requirements for this project embrace all aspects of Geotechnical Practice and clearly illustrate the breadth and capacity of geotechnical work undertaken by railroads in Canada.

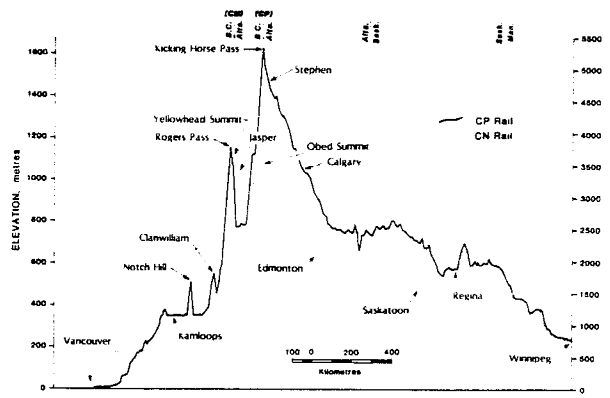


Figure 7: CN Rail and CP Rail profiles between Winnipeg and Vancouver.

THE CP RAIL ROGERS PASS GRADE REVISION

The unique requirements of railroad grade in comparison to that of highways and the special circumstances of Rogers Pass are illustrated in Figure 8. The Pass is a narrow valley that crosses the Selkirk Mountains. Its elevation is more than 1,200 m while the elevation of surrounding peaks is over 3,300 m. The average annual snowfall in the Pass often exceeds 10 m. Avalanches are a widespread problem that plagued the original railway line which was built along the route selected by Colonel Rogers through the summit of the Pass. To avoid avalanches and eliminate many of the curves on the original rail route, the Connaught Tunnel was built under Rogers Pass. The tunnel, completed in 1916, is 8 km in length, and reduced the summit of the rail line by 165 m, as shown in Figure 9. The current grade revision reaches Mt. MacDonald about 100 m below the elevation of the Connaught Tunnel. The Trans Canada Highway which is not constrained by aspects of curvature or gradient easily ascends the Pass as shown in Figures 8 and 9.

Selection of the route shown in Figure 9 was undertaken in the mid 1970's by Geotechnical Engineers from CP Rail and several consulting firms. This involved extensive engineering geology mapping and limited subsurface exploration. Routes on both sides of Beaver Valley (Figure 9) as well as various tunnel locations were considered. Geotechnical conditions related to natural slope instability and ground conditions for tunneling were among the important factors that controlled route selection. Operational advantages and selection of a route which could be built with the least disturbance to the environment of Glacier National Park, one of Canada's most pristine wilderness areas, were other principal factors.

The grade revision is 33.8 km long and includes the following main components:

1. Surface Grade:
   - 17.3 km of line
   - six bridges
   - three concrete box culverts
2. Mount Shaughnessy Tunnel
3. Mount MacDonald Tunnel
4. Ventilation Shaft

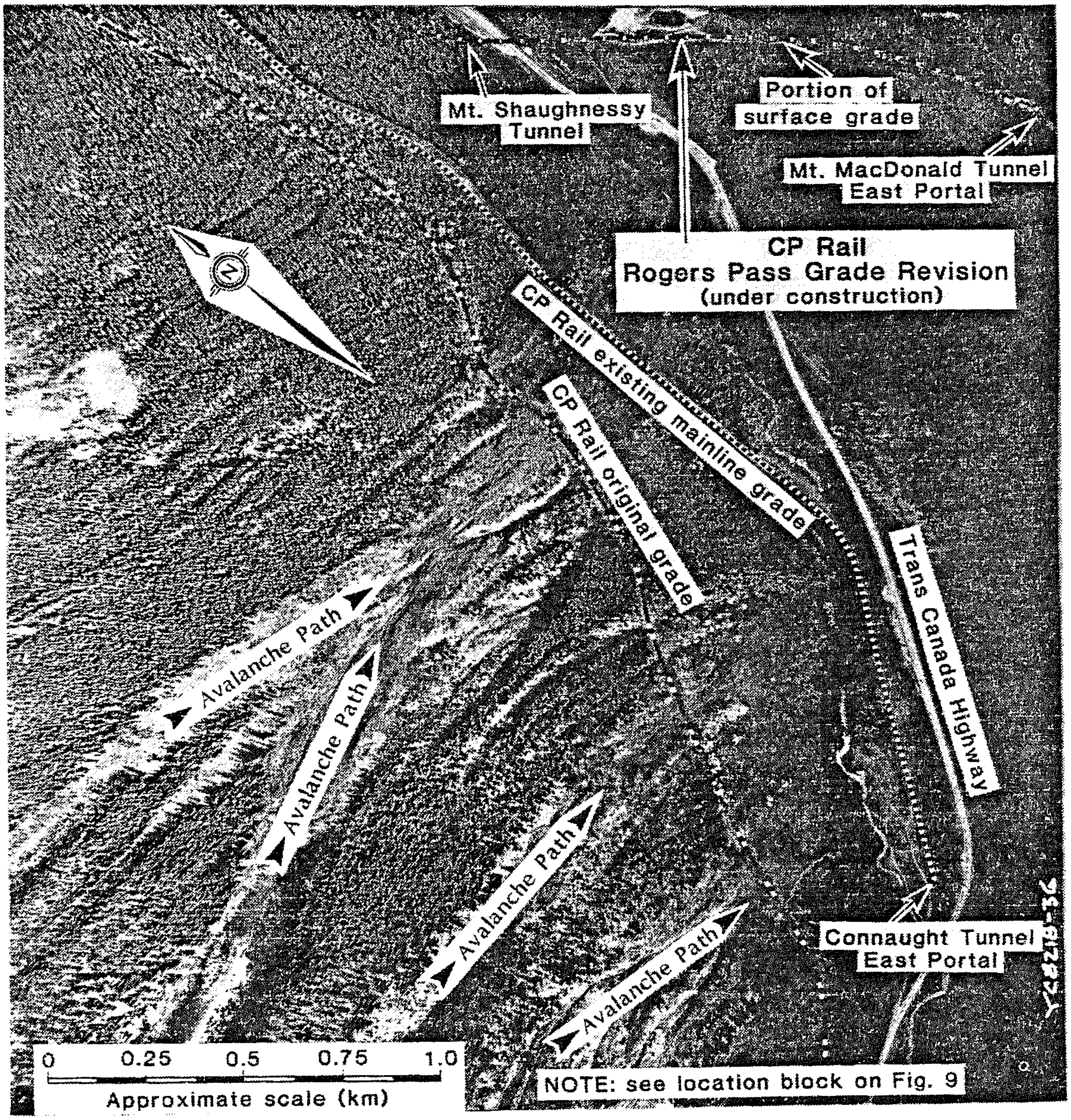


Figure 8: The entrance to Rogers Pass in the Beaver Valley.
- From Kenting Earth Sciences air photo YC 8218-36 (Scale 1:12,000) with annotations.

Surface Grade

A geotechnical investigation was undertaken along the entire length of the surface grade. This began with detailed terrain analysis which was used to plan subsurface exploration.

Several different types of drills and a back-hoe were used for subsurface studies. All worked on a specially prepared access road along the new right-of-way. Soil and rock logging, sampling and coring, in situ tests, and instrumentation monitoring of ground water and slope movements were undertaken in drill holes. Engineering geology mapping and sampling were completed in all soil and rock cuts along the access road. A variety of geotechnical testing was carried out on soil and rock samples. The results were compiled and analyzed to produce design recommendations for earthworks, borrow sources, drainage and earth retaining structures.

The surface grade begins on gravel terraces at Rogers siding (Figures 8 and 9) but quickly moves onto mountain slopes to begin its 1% climb. For the entire route along Beaver Valley it runs parallel to but below the existing mainline. The mountain slopes are made up of varying thickness of surficial deposits overlying bedrock. Surficial soils range from gravel till to stratified gravels and sands comprising moraine and deltaic or fluvial deposits, respectively. Bedrock consists of phyllites, schists and quartzites

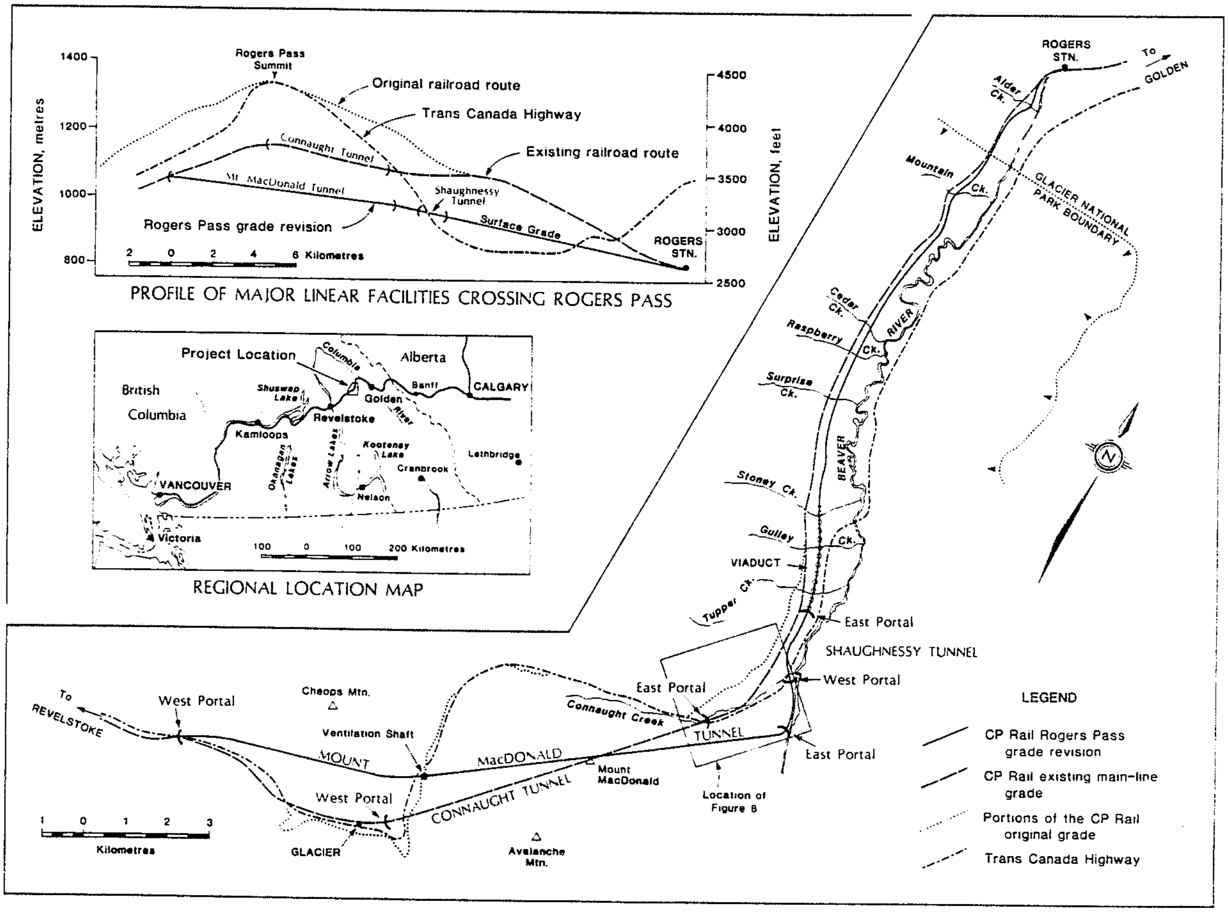


Figure 9: Plan and profiles of the Rogers Pass Grade Revision project showing the original, existing and new grades and the Trans Canada Highway. This includes a regional location map and a location block for Figure 8.
- From CP Rail base map and data taken from NTS topo sheets for Trans Canada Highway profile.

which dip steeply into the slopes.

Initially the cross-slope is suitable for conventional cut and fill or unsupported rock excavation. As it steepens, a total of 26,000 square metres of retaining structures and occasional bridges are used intermittently with conventional cut and fill. Ultimately the cross-slope becomes so adverse as to make conventional cut and fill as well as retaining walls prohibitive from the standpoints of engineering feasibility and aesthetic impact on the environment of a National Park. In this extreme cross-slope the grade is built on a 1,229 m long viaduct consisting of 44 piers and two abutments. The piers are cast-in-place and support 27.4 m orthotropic steel spans. Foundations for the piers and abutments consist of pencil piles and anchors.

An evaluation of stream flow and debris flow potential was made at all major crossings. A potential for debris flow was identified at six locations but three of these already had high-level bridge crossings with adequate clearance. Debris flow restraint structures and associated concrete approach channels are built at the remaining three crossings. Foundation conditions for most bridge and debris torrent structures consist of gravel with some rock at a few locations. Footings are used for most of the foundations. One of the bridges required extensive rock bolting to prevent wedge movement beneath the abutment. Soft glaciolacustrine soils at the lower end of Beaver Valley required pile foundations for one bridge. Five 3.1 m diameter CMP culvert sections were used as an alternative to a bridge at another site.

The surface grade also crosses two major bedrock landslides. Large fills are used to minimize cuts which would have a destabilizing affect on these mass movements. Improvements to surface drainage between the existing and new grade were also completed as part of construction in order to improve overall slope stability.

Mt. Shaughnessy Tunnel

At the west end of the viaduct structure in Beaver Valley, the Trans Canada Highway is immediately below the new rail line. The highway is in a rock cut with a long history of slope instability related to toppling. Because of the extremely steep cross-slope, and the potential for slope stability problems related to extension of the viaduct structure and to an overpass of the Trans Canada Highway, the alignment was moved into the Shaughnessy Tunnel. The tunnel is 1.85 km long and extends from the west end of the viaduct structure to a portal location after passing beneath the Trans Canada Highway.

Geotechnical investigations for this tunnel included drilling and surface mapping. Drilling was undertaken at accessible points along the alignment and in the portal areas. Rock cores and soil samples were obtained and tested. Instrumentation was installed to measure groundwater levels. Detailed engineering geology mapping of the Trans Canada highway rock cut was used with the drilling and instrumentation results to prepare the tunnel design.

The Shaughnessy Tunnel is located in strongly foliated phyllites, schists and quartzites of the lower Horsethief Creek Formation. The foliation dips steeply into the slope and trends parallel to the tunnel. Foliation controls the rock mass properties. Rock bolts and shotcrete were generally used for both temporary and permanent support. Steel sets and concrete were used in soft ground and poor rock. More than 300 m of softground tunneling were needed to expose rock faces at the portals. Heavy support with steel sets was used through these softground sections. The tunnel designs are shown in Figure 10.

Mt. MacDonald Tunnel

Geotechnical investigations for the Mt. MacDonald Tunnel extended over a five year period. They included engineering geology mapping and subsurface drilling investigations.

The mapping program included detailed examination of select surface exposures and of the unlined Pioneer Tunnels, driven during construction of the Connaught Tunnel. A detailed geological map and the cross-section shown in Figure 11 were prepared from the mapping data. This showed the rock types and principal structures expected along the tunnel heading. These data were used to plan the subsurface drilling program.

The subsurface program involved drilling from accessible locations along the surface trace of the tunnel alignment and at select locations where major structures could be investigated with substantially reduced cover. More than 5,000 metres of rock drilling was conducted for the project and the vast majority of these were for the Mt. MacDonald Tunnel. Instruments to monitor groundwater level were installed in the portal areas. Comprehensive laboratory testing was conducted on the rock core to determine strength properties.

Phyllites, schists and quartzites are in equal abundance in the eastern portion of the tunnel while quartzites predominate in the western portion (see Figure 11). The contract for driving the tunnel was therefore broken into three sections, viz. the east and west ends, and the central portion together with underground facilities for the vent shaft. A tunnel boring machine was used for the top heading and conventional drill and blast for the bench in the east end contract.

Conventional drill and blast in a full face operation was used for the west end and central contracts. The finished tunnel is continuously lined with a cast-in-place concrete liner. The tunnel design is shown in Figure 10.

Tunneling conditions were good compared with those of the Shaughnessy Tunnel because the foliation is perpendicular to the tunnel alignment. Nevertheless scaling and temporary support with bolts were necessary on the majority of the west heading with sporadic temporary bolts installed in the east heading. A combination of open-cut, softground and mixed face tunneling was necessary over 480 m to expose the rock surface at the west portal. The east portal required an excavation through 10 m of colluvium and till and 24 m of rock. The overburden was supported by a tied-back soldier pile and lagging wall with rock anchors used to stabilize the rock excavation.

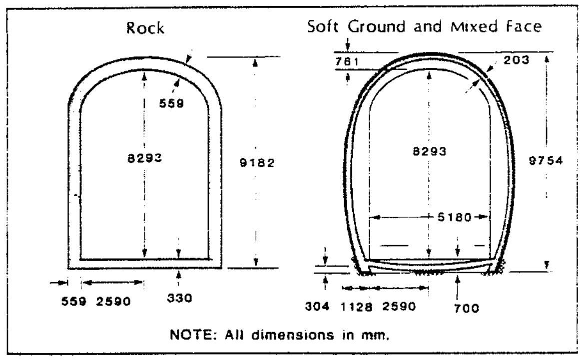


Figure 10: Geological cross-section along the Mt. MacDonald Tunnel.

Ventilation Shaft

Westbound trains passing through the Mt. MacDonald tunnel require fresh air for cooling and to purge the exhaust air from the tunnel after the train passes through it. In order to purge the tunnel with fresh air fast enough to realize the necessary train frequency, a central vent shaft is necessary. This is designed to allow the east half of the tunnel to be purged while a train continues through the west half.

The summit area of Rogers Pass is approximately the central point of the tunnel and hence became the required location for the vent shaft. Four locations were considered. One was eliminated because of its visual impact, another because of avalanche danger and a third because of a massive rock slide. This left the location shown on Figures 9 and 11 which was about 1,000 m south of the straight line surface trace and required that the tunnel alignment be curved slightly to connect with the shaft.

Geotechnical investigations included drilling and coring the full length of the shaft. Several additional holes were drilled to facilitate groundwater level monitoring and hydrogeological testing of aquifiers in the

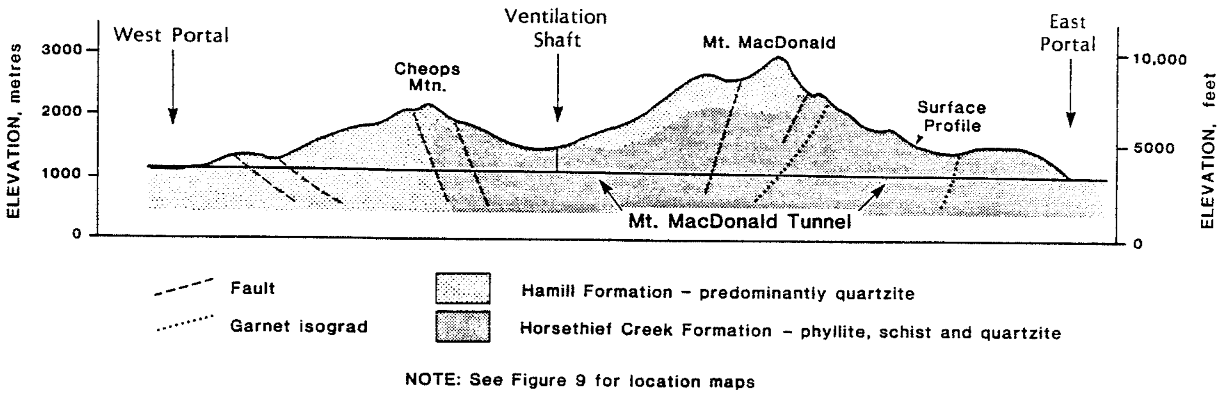


Figure 11: Mt. MacDonald Tunnel design cross-sections.

overburden. Extensive laboratory testing was also undertaken to facilitate shaft design.

The site area required levelling by excavation and minor filling, including construction of a 15 m high segmentally excavated anchored retaining wall. The 349 m deep 8.5 m diameter shaft was sunk conventionally. It was necessary to use ground freezing to penetrate materials below the water table in the lower portion of the overburden section.

Virtually all of the geotechnical engineering requirements of this megaproject were handled by CP Rail Engineers or Geotechnical Practitioners in consulting firms across Canada. The project is nearing completion at this time and is currently on schedule and on budget.

CONCLUSION

In celebrating the centennial of the Engineering Profession in Canada it is appropriate to recognize the long and important role of Geotechnical Practice in railroad transportation. The geotechnical challenges that confronted engineers involved in early railroad projects were immense and in some cases without precedent. These were surmounted with technical innovation often many decades ahead of its time. The same spirit accommodated new geotechnical challenges through the period of rapid growth in railroads and continues today with megaprojects like the CP Rail grade revision project in Rogers Pass, British Columbia.

Railroads will continue to play a key role in the future of Canada. The predominantly east-west system of today may only be a temporary visage of an ultimate mosaic which may one day cover all of Canada. Railroads are already slowly extending north from several locations into Canada's, and indeed, one of the world's last frontiers. The geotechnical challenges associated with this future growth and with maintenance of the existing system are impressive. Fortunately, Geotechnical Practitioners in Canada have a strong heritage in railroad applications, and hence confidence in their ability to meet future challenges.

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Geotechnical Practices in Waste Management

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SUMMARY

This paper provides a review of the role of geotechnical practice in waste management. Prior to 1970, geotechnical practices were seldom applied to sanitary landfill or waste management projects. However, with the advent of environmental concerns and the subsequent legislation associated with them, geotechnical techniques began to be used on a regular basis. Landfill design became more complex and sophisticated. The concept of the fully engineered landfill with its related liner systems, leachate collection and treatment designs has become accepted practice. The need to deal with special wastes and the option of containment in deep clays has necessitated the need to deal with excavation slope stability problems, bottom heave and settlement. Geotechnical practices are also used in the remedial designs for mitigating older, leaking landfills and abandoned waste disposal sites. Although many of those measures appear to be hydrogeologically oriented, their investigation and proper and effective design necessitates the application of geotechnical expertise. As containment measures and mitigative systems become more sophisticated in the future, so will the requirements of geotechnical practices for the innovative solutions to many site specific waste management problems.

RÉSUMÉ

Cet article passe en revue le rôle de la pratique géotechnique dans la gestion des déchets. Avant 1970, les pratiques géotechniques étaient rarement appliquées aux projets de gestion des dépotoirs et des déchets. Cependant, suite aux préoccupations environnementales, et aux lois qui ont suivi, on s'est mis à utiliser régulièrement les techniques géotechniques. La conception des dépotoirs devînt plus complexe et sophistiquée. Le concept du site d'enfouissement totalement conçu par des ingénieurs, avec parois d'étanchéité, système de drainage du lixiviat et systèmes de traitement, est passé dans la pratique. Parce qu'on a choisi de confiner certains déchets spéciaux dans des dépôts profonds d'argile, il a fallu faire face à des problèmes de stabilité des parois d'excavation, de soulèvement du fond de fouille et de tassements. Les pratiques géotechniques sont aussi utilisées quand il faut concevoir des solutions pour atténuer les problèmes dus à des anciens sites de déchets à l'abandon avec des fuites. Même si bon nombre de ces mesures semblent être hydrogéologiques, on a besoin de l'expertise géotechnique pour les investigations ainsi que pour une conception particulière et efficace. Dans le futur, quand les méthodes de confinement et les systèmes d'atténuation deviendront plus complexes, il en sera de même des exigences géotechniques pour des solutions originales à des problèmes spécifiques de gestion des déchets.

INTRODUCTION

Geotechnical practices in waste management only really began in the late 1960's in Ontario and elsewhere in Canada. Since that time, geotechnics has played an increasingly important role. This paper deals with the historical development of geotechnical practice in waste management and conversely with the influence of waste management demands on geotechnical practices and techniques.

Specific areas of concern, the need for research, and the future role of geotechnical practice in waste management are discussed. What becomes apparent in this review is that geotechnics must be applied in co-operation with other disciplines. The geotechnical engineer must become more cognizant of the need and type of input that these other disciplines, such as hydrogeology, organic chemistry, toxicology and process engineering can contribute to the solution of what were once thought of as only geotechnical concerns.

On the other hand, the geotechnical engineer must make the other disciplines aware of the types of input that can be provided to achieve the desired results. Because of the demand of the market place, this part of geotechnical engineering is rapidly expanding. In our opinion it will continue to expand in the forseeable future.

The types of waste are both natural and man made and are highly variable in their chemical and physical make up. In order to properly deal with the management of these wastes the geotechnical engineer will face a complex future.

A DECADE OF CHANGE - THE 1970's

Prior to 1970, there was little input of geotechnics into waste management practice. Solid wastes were relegated to municipal dumps placed in an uncontrolled manner upon derelict lands, such as abandoned gravel pits or floodplains. The geotechnical engineer's concern with waste revolved around the sewage treatment plant and its soil mechanics' problems.

But with the advent of the 1970's the public became aware and concerned with the effects of waste disposal on the environment. The public pressured the politicians and the politicians looked to the engineers for solutions. The result was that waste management practices were forced to change, and the geotechnical engineers found themselves in the midst of social change. These changes required increasing degrees of technical input, as the waste facilities evolved from dumps of the past to engineered landfills, complete with leachate collection systems, and containment mechanisms. At the beginning of this change the engineers found themselves coping with new problems that required much more than their past training had prepared them for. In the past, geotechnical engineers handled ground water problems as they related to excavations and pore water pressures. They were not always equipped to understand the complexities of contaminant flow and modern ground water theories.

In the 1970's hydrogeology played a lead role in this field, especially in site selection and leachate management system designs. It was found, however, that the design and construction of such components as liners, where they were associated with low permeability soil materials, required geotechnical specifications and emplacement techniques that were beyond the expertise of hydrogeologists. This input of geotechnics had reached an increasingly important role for landfills in clays where slope and cell base stabilities, optimization of loadings to prevent distortion of underlying collection systems and cover materials, were key to the successful design of the facility. Thus, the requirements of society were demanding a blend of both hydrogeology and geotechnics to solve the environmental problems posed by waste management techniques.

The Environmental Assessment Act of Ontario required that all of the alternatives to any undertaking be examined and assessed. These considerations meant that sites in less environmentally safe settings must also be considered and evaluated. This necessitated a greater emphasis on the safe engineering of such facilities, and hence a greater role for the geotechnical engineer.

In the site selection phase, hydrogeology played an important part in the process. Using terrain evaluation techniques, the scientist searched for locations where nature provided natural containment. Geotechnics usually played little or no part in the selection process. Its role was relegated to design only. As the waste facilities became associated with soft clays, however, the problems of deep cell excavations, entombment and perpetual care required the skills of the geotechnical engineer.

Due to the increasing emphasis on designed landfills, there was the need for more engineering including involvement into mitigative measures, for old established landfills. These measures included leachate control systems, cut off walls and gas interceptor trenches. In these fields the hydrogeologist often began with the lead role, with little geotechnical input prior to construction. For example, leachate purge wells were usually designed solely on the basis of the geometries of cones of capture and yields. There was little thought of the potential consequences due to the change of the effective stress of the soils. This could result in undesirable consolidation and settlement of nearby foundations and buried services. As the profession gained experience, we began to see an increasing use of the multi-discipline approach to waste management problems, with the geotechnical engineer working together with the hydrogeologist. Thus, the 1970's were a decade of learning and change. In the early 1970's, the hydrogeologist took the lead, but by the end of the decade, there was an increasing awareness of the contributions of the geotechnical engineer.

DECADE OF EXPANSION - THE 1980's

In the 1980's the horizons of both the engineer and the geologist are quickly expanding, as the chemical impacts of the wastes on both the environment and man are being scrutinized. The problems of safely disposing of organic compounds present a whole host of new technology. In recent years there has been an emphasis and focus on industries where such chemicals have been produced along with undesirable wastes and residues. Until recently, some materials were considered "inert", but such is no longer the case. Unfortunately, many of the old industrial facilities were not designed to contain the wastes. In fact, in some cases the wastes were used to fill in ravines and lowlands and in other cases to actually reclaim land. Many industries are now assessing their past disposal practices and investigating the need for mitigation and decontamination of these old sites. Many of our standard geotechnical practices will require modification and innovation to deal with these challenges. We are now sampling and testing complex organic chemicals in solid, aqueous and gaseous phases, where concentrations are often measured in parts per billion or trillion, and the materials are not degradable. Thus, field drilling techniques must be designed to prevent cross contamination between boreholes, and to obtain samples in a variety of media. The wastes may be adsorbed onto the soil solids and/or be in fluid to semi-fluid form with both miscible and immiscible phases. These non aqueous phase fluids may be both more or less dense than water and contain volatiles.

In some of the old plant areas, conventional geotechnical investigations were carried out solely to deal with foundation aspects of the facilities and services. At that time little thought was given to the need for special measures concerned with the closing and sealing of boreholes. As a result, boreholes that were simply backfilled with the materials off the flight augers may provide an artificial pathway for wastes, through what was originally considered to be a very low permeability clay barrier between the surface and an underlying aquifer.

Decommissioning of obsolete plants is now common place. Some of the measures associated with these cleanups require the input of geotechnics to achieve the desired results without creating adverse, unexpected affects on adjacent properties or on the decommissioned property. One special type of problem that is evolving is the clean up of old forgotten plant sites that have been built over by redevelopment of the site. An example of this is the old coal gasification plants that once were common throughout Ontario. These plants were generally located near rail facilities or docks in the major urban areas of the day. In many cases, when the usefulness of these plants ceased earlier in the century, the toxic residues, tanks and other equipment were simply left in place, often to be buried.

The restoration of these sites, including the excavation, recovery and ground restoration will require the expertise of not only the geotechnical engineer, but the hydrogeologist, the process engineer and the environmental chemist. These areas of expertise will be of particular importance where these plants are located in urban areas where excavation and fluid recovery are adjacent to other buildings and services.

Another example of a 1980's problem where geotechnics will play an increasing role is that of leaking underground storage tanks. Although this is not usually considered as waste management, it is a situation were a waste has been created in an unplanned way. The solutions may involve excavation and/or fluid recovery, again in an urban setting. To date, this has involved only minor geotechnical input, however, this problem will become more commonplace in the future. Many service stations have been located in downtown areas where property prices have escalated the value of the stations and increased the costs of maintaining them. Because many of these stations are old there is the increased danger of leaks and underground contamination. Decommissioning of these lands will require recovery techniques that include proper geotechnical input and practice. In the past, most geotechnical investigations on properties proposed for sale and development were designed solely for foundation aspects. At the present time and increasingly in the future, these investigations will have to consider contamination and waste management concerns as well.

GEOTECHNICAL RESEARCH-NEEDS FOR THE FUTURE

There are several areas of geotechnics where research is desperately needed as it relates to waste management. These areas range from field techniques, to laboratory testing, sampling and analysis.

In the planning of field investigations for waste management work, several other aspects have to be considered along with the geotechnical objectives. One of these is the understanding of the wastes themselves. Both the physical and chemical natures of the wastes must be examined prior to the field operations proceeding. The characterizations of the wastes are necessary so that not only can the sampling program be properly planned, but also for the development of safety protocols to protect field personnel. The safety of all concerned is a crucial factor and cannot be ignored. Special equipment such as air packs, proper disposable clothing and protective footwear are often required on site. Arrangements for the after use disposal and/or storage must be considered before the field work begins. Needless to say, these safety precautions must also involve the subcontractors, such as the drilling crews, and must extend to not only the people, but also the equipment used on the site. Measures must be taken to ensure that the drilling equipment does not carry remnant contamination from previous investigations before it is used on site. Protocols and techniques must be developed to ensure that contamination is not spread from hole to hole, or artificially to depth. And when the equipment leaves the site, protocols have to be established to ensure that the equipment does not contain even trace amounts of contaminants that might adversely affect the next project that the equipment is used on.

As well as the characterization of the waste itself, we must consider the overall geometry of the waste deposit. This may involve the integration of information from several sources;

(i) the stereoscopic interpretation of sequential historic airphotos,

(ii) topographic mapping, both present and historical,

(iii) industry records and interviews of plant personnel,
(iv) records and reports of previous studies,
(v) site plans and servicing details
(vi) the use of geophysics, and
(vii) the review of old assessment documents.

This background will provide a rationale for the proper placement of boreholes and the sampling strategy. The characterization of the waste will also provide a basis for the selection of proper instrumentation to be left as monitors in the boreholes and for borehole annulus sealing. This is especially important in the case of organic contaminants where laboratory testing must deal with concentrations in parts per billion, or even per trillion. Even the use of glues for instrumentation and casing can create artificial results. As well, certain contaminants may react with the monitor materials either in the form of leaching or adsorption.

In some cases it may be necessary to provide more than one set of augers so that the work can continue without delay from hole to hole and to depth. One set of augers can proceed with the drilling while the other is being decontaminated. In extreme cases, protocols may dictate that on site laboratory testing be available to ensure that the equipment is completely clean. Such studies may also require that the investigation be planned in a phased program, where the waste is dealt with first, before each succeeding lower unit to depth is sampled.

In many natural containment sites the soils are fine grained sediments or glacial tills. The weathering processes have created a surface fracture zone which in turn has created a zone of secondary permeability, hence a potential pathway for contaminant migration. The unfractured clays below the weathered crust still maintain their primary permeabilty so that contaminant migration is primarily controlled by diffusion. In any investigation it is important to properly characterize the low permeability barrier and also to prevent any contaminants from being carried downward artificially during the drilling procedure. One method of doing this is to use a phased program of investigation. In such cases a large diameter hole is drilled through the fracture zone down into the intact clays below, and grouted with a bentonite cement mix and allowed to set. Hollow stem augering and sampling using standard diameter flights can then safely proceed to depth. Often it is wise to use separate boreholes to sample the different units, such as the weathered crust, the intact clays and other geological units below.

Sampling wastes themselves can be one of the most critical tasks, since the wastes may range from a heterogeneous mix through to a sludge. This is an area where future research and development is needed to develop techniques that will allow the recovery of representative samples. In such work some of the techniques developed in tailings investigations may be applicable. Often, standard split spoon and Shelby tube sampling are not adequate. Something similar to the continuous coring techniques developed for natural soils may have to be applied to the sampling of these heterogeneous wastes.

The geotechnical engineer and the hydrogeologist use similar techniques and they measure many of the same parameters in situ. Many of these techniques have become standard and are well documented, while others require the need for research. One of these important parameters is the measurement of in situ permeability in several situations;(i)in unsaturated materials above the water table, (ii)in fractured weathered zones of surface soils, (iii)in fractured bedrock, and(iv)in very low and very high permeability materials, the extremes of the scale.

Research has begun in several areas, especially in the area of fracture flow. Some of the techniques applied to rock in the nuclear waste programs both in North America and Europe have begun to be applied in soils. Problems relating to the sidewall smearing of fractures caused by the drilling and sampling procedures are currently being addressed. In situ testing of soils in the future may involve packer systems and transient testing. This is an area where the hydrogeologist and the geotechnical engineer must work together, rather than along separate paths as they have in the past. Analysis of the effects of the fracturing on permeability is not only hydraulic in nature, but it is also related to the state of stress of the subsurface materials and to chemistry as well.

As with other geotechnical practices, field measurements and parameters are usually correlated with laboratory tests carried out on the recovered samples. Geotechnical laboratory testing procedures have been well documented. Some of these parameters, however, have been shown to be affected by some chemical compounds. Some highly concentrated solvents have had adverse effects on the permeability of clays. Unfortunately, the effects of various concentrations for a variety of wastes and leachates have not yet been established in terms of techniques or results. The importance of this in the field of artificial liner assessment cannot be underestimated.

Laboratory test techniques and protocols need to be established for a number of wastes, recognizing that the pore and fracture fluids will be leachate rather than water. The tests should be designed to simulate field conditions. The aspects of the testing of fractured materials in the laboratory also needs to be addressed. More research into the correlation of geotechnical properties, chemistry, and clay mineralogy appears to be necessary.

THE FUTURE

Future papers on geotechnics and waste management will undoubtably include case histories. At the present time, however, there is not the history available to confirm the success of the design parameters or their long term performance. The design and analysis components of geotechnical practice applied to waste management has been used only during the past decade or two. The emphasis has been placed mainly on the construction of clay liners. Other geomechanic techniques are directly applicable to the construction of waste cells, such as stability of the cell slopes and bases.

The design of landfill covers requires the blend of both hydrogeological and geotechnical theories and practices, and this is one area where future research is required. Proper design of final covers will minimize both infiltration and future leachate production. However, in the overall management of some facilities it may be advantageous to promote rather than impede such leachate production. In the past, covers have involved the placement of clays considered by some to be almost impermeable. In fact, clays develop a secondary permeability due to fracturing. Research of layered covers with complexing of various permeability components has already begun. Unfortunately, in many cases the geotechnical feasibility has not been considered, and in the future a more holistic approach must be used.

In the past, containment and leachate collection systems have been designed mainly on hydraulic parameters. Geotechnical designs, especially for deeper cells in soft clays to ensure minimal deformation of the system, will have to be developed in the future.

Another issue of the future that will affect geotechnics in waste management is not technical, but legal. Present trends of the insurance companies are to exclude professional liability insurance coverage on those projects that involve pollution or the threat of pollution. At this time, work involved with contaminants has a high exposure and professional liability insurance is becoming difficult if not impossible to obtain. While the insurers and professionals wrestle with these liability problems, it appears that only those aspects actually related to design may be covered, while those pertaining to pollution clean up as a result of an error, may not be covered. When you combine this liability exposure with the complexities of the technical problems and the traditionally competitive nature of the consulting geotechnical engineer, it seems predictable that there will be a thinning of the ranks of those offering geotechnical services in waste management. Indeed, this issue of liability may in fact be one of the most important factors facing geotechnical practices in waste management in the future.

An Evaluation of the Canadian Nuclear Fuel Waste Management Program

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SUMMARY

The Canadian Program on Nuclear Waste Management conducted by Atomic Energy of Canada Limited is reviewed. The concept involves a multi-barrier system and includes a 1 km deep vault in the Canadian Shield. Results from current research is anticipated to be formally reviewed in 3 years. The complex assessment process includes public hearings, and precedes any site selection. The role and work of the Technical Advisory Committee, established as an independent scientific and engineering review body is described.

Extensive geological, geophysical and hydrogeological work has been carried out in the past eight years. A major regional flow system study has been initiated. Predictive aspects are being tested through the construction of an Underground Research Laboratory (URL) in the Lac du Bonnet batholith. The URL is a key component in gaining the necessary scientific understanding of the type of rock system that could eventually be selected somewhere in the Canadian Shield to host a waste disposal vault. Results so far achieved justify this endeavour, and show the importance of the recent Canada-U.S. agreement for U.S. participation in extending the URL.

The methodology being employed to assess the environmental and safety outcome of a disposal vault is based on a probabilistic approach and is considered sound.

INTRODUCTION

The research and development program to establish appropriate technologies for the safe, permanent disposal of Canada's nuclear fuel wastes was formally initiated in June 1978 by a Federal Government-Ontario Agreement (Canada-Ontario, 1978). The resulting Canadian Nuclear Fuel Waste Management Program (CNFWMP) is described in a public document (Boulton, 1978) and its course of development can be followed through a series of publicly available annual reports (e.g. Dixon and Rosinger, 1984). External peer review of the program is made annually by a Technical Advisory Committee (e.g. TAC, 1986).

THE CANADIAN WASTE DISPOSAL CONCEPT

The Objective

The overall objective of the CNFWMP is "to ensure that there will be no significant adverse effect on man or the environment from nuclear fuel waste at any time" (Rosinger and Dixon, 1982). Nuclear fuel waste is defined as either used or irradiated fuel, or high-level radioactive material separated from used fuel through reprocessing (should Canada adopt this option in the future).

The Multi-Barrier System

Similar to efforts in several other nations, the Canadian program of research on deep geological disposal of nuclear waste is based on the concept of isolating wastes with a series of barriers situated in a deep (500 m - 1000 m) underground vault, built in a stable, terrestrial geologic formation such as the Canadian Shield. This multi-barrier concept is illustrated in Figure 1.

First, the bundles of used fuel would be encased in canisters with an anticipated minimum lifetime of several hundred years which correspond to the period of high fission-product activity. The containers would be designed to withstand vault pressures and be resistant to corrosion under the temperature, groundwater exposure and radiation fields that could potentially exist.

Research is also being conducted on immobilizing high-level wastes from fuel reprocessing (if adopted) whereby the waste would be incorporated into a water-insoluble, leach-resistant material, such as glass or ceramic.

Following waste immobilization or containment, the canisters would be emplaced in the vault and packed with compacted buffer material such as bentonite clay. This candidate material swells upon contact with water, thus acting as a seal against leaching and corrosive agents. It also has a high capacity to absorb chemical species, including most of the significant radionuclides. The vault and the shaft would then be backfilled, probably with a mixture of sand and bentonite, to close any opening to the surface.

The geologic medium in which the underground vault is built acts as yet another barrier to the migration of radionuclides should they escape the vault. Retardation occurs as a result of a number of natural processes taking place at depth, including chemical absorption of radionuclides onto rock surfaces, ion exchange, diffusion into the body of the rock, and the long path lengths to the surface due to the relatively small size and frequency of fractures in the chosen rock. The effectiveness of this natural barrier depends on careful selection of a site which exhibits favourable geochemical, geological and hydrogeological conditions.

Finally, although the biosphere is not generally regarded as a "barrier" in the same sense as those described above, the surface environment, with its large volume of soil and water, has a great capacity to disperse and dilute whatever material which, no matter how unlikely, may reach the surface.

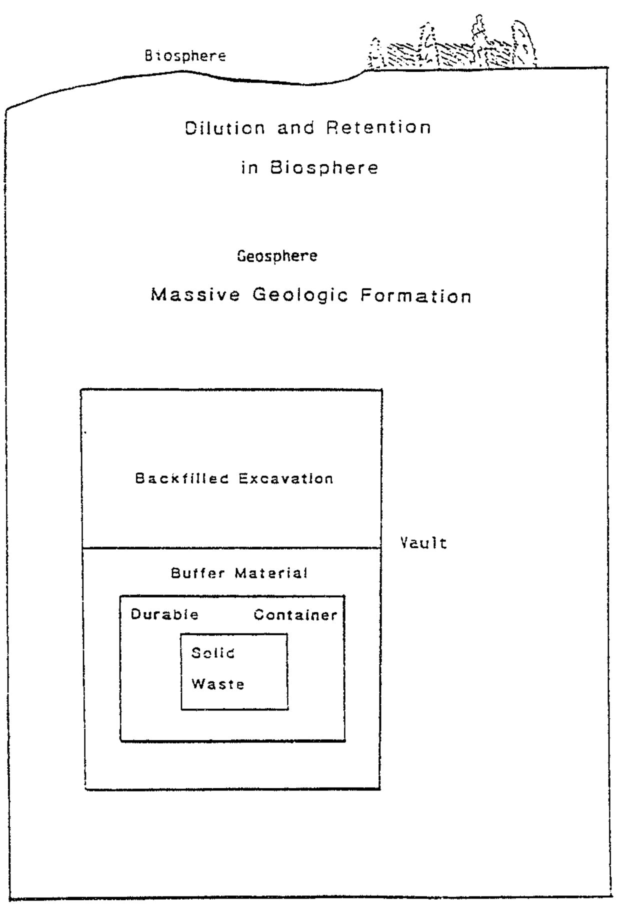

Figure 1: Schematic of Natural and Engineered Barriers to Migrating Radioactive Waste

Four Parts of the Research Program

The research work in support of the disposal concept is divided into four parts:

1. interim storage,
2. transportation of nuclear fuel wastes,
3. immobilization of wastes, and
4. disposal or burial of wastes.

R&D work on the first two parts is referred to as "Pre-Closure Assessment" and is the responsibility of Ontario Hydro (OH). Atomic Energy of Canada Limited (AECL) has the responsibility of implementing research on the last two parts known collectively as the "Post-Closure Assessment". This work includes the assessment of possible safety and environmental impacts from the disposal vault, both in the time period during its operation and also far into the future after its closure.

Major Components of the Post-Closure Assessment

The Post-Closure Assessment comprises research in a multitude of disciplines. Components of the program may be grouped under four headings: geoscience, engineered barriers, bioscience and system analysis. Included under the first heading are hydrogeology, geology, geophysics, geomechanics and geochemistry. Engineered barriers include immobilization of wastes with the use of glass or ceramics; containers (composition and design); buffers and backfill. Under bioscience, studies include radiologic effects, dosimetry, ecological succession, limnology, hydrology, plant uptake, and food chain models. The objective of system analysis is to process and integrate the results from all of these disparate studies and present a comprehensive and comprehensible statement about the safety and acceptability of the disposal concept.

The Three Phases of the Program

The first phase of the program involves the assessment of the concept that the disposal of immobilized fuel or wastes in deep, stable, terrestrial geologic formations can be achieved with the stated objective of safety and permanence. This "generic research and development phase" spans the period 1981-1990, but builds on many years of prior research. If the concept is deemed acceptable sometime during this period, the second phase, that of site selection involving the actual process of locating a suitable site, could follow. The final phase would be the construction and operation of a demonstration vault which would provide the engineering and operational experience for a fuel repository sometime in the next century.

EVALUATION OF THE CONCEPT

The 1981 Canada-Ontario Statement

Concept assessment, as the first phase of the program, involves generic research on geological disposal being conducted by AECL, OH and other parties, that will provide the evidence upon which the acceptability of the concept will be evaluated. In a joint statement the Ontario and Federal Governments defined, in detail, the process for that evaluation. Public announcement of the joint statement constituted the first step of that process. In that document (Canada-Ontario, 1981), the Atomic Energy Control Board (AECB) is designated as the lead agency for the regulatory and environmental review of the disposal concept, and is responsible for the issuance of a final regulatory statement (AECB, 1985) which outlines the basis for such a review.

The Concept Assessment Document

The focus of this review will be a Concept Assessment Document (CAD) submitted by AECL. Presently this document is undergoing revision based on new scientific data and results, and also on feedback received from various parties on the two interim documents issued by AECL in 1981 (Wuschke et al.) and 1985 (Wuschke et al.).

The final CAD is scheduled to be completed during 1990. Upon completion, the CAD will be submitted for regulatory and environmental review to the Interagency Review Committee (IRC) composed of AECB, the Ontario Ministry of Environment and the Federal Department of Environment. It will also be sent to other government departments and offices across Canada, to public libraries and will be available to interested parties, the general public and the scientific community.

The Regulatory Review and Public Hearing Process

Announcement of the CAD submission will initiate the process of regulatory review by the IRC as well as the process of conducting a public hearing. IRC will issue a public report after its review of CAD. It appears likely that the public hearings will be conducted by a review panel under the auspices of the Federal Environmental Assessment and Review Office (FEARO), with referral of the project to the Minister of Environment Canada, presumably sometime in 1987.

The process calls for the Minister to appoint a review panel with specific terms of reference (See Figure 2). Usually the first task undertaken by the panel is to hold general meetings with the public to obtain input on what the issues are, and to then issue draft guidelines describing what the detailed review process will address. If this procedure is followed, comments from the AECB, AECL, OH, Provincial and Federal Government Departments and the public would be taken into account, and a set of final guidelines issued. Upon receipt of AECL's CAD, the panel would conduct a review and, with the IRC's report on CAD, would determine if the guidelines and regulatory criteria had been met. Revisions would then have to be made by AECL if there were deemed to be deficiencies, otherwise the panel would announce, prepare for, and conduct public hearings sometime during 1991. The panel would report its findings to the Minister of Environment Canada and the Minister of Energy, Mines and Resources Canada, who would make them public and initiate a departmental review of the panel's recommendation.

It will then be the responsibility of the Federal and Ontario Governments to make a decision on the acceptability, conditional acceptability or non-acceptability of the concept presented in the CAD.

Actual site selection for a waste disposal facility (the second phase) will commence only after the concept has been accepted. Should the concept be conditionally accepted, further research work by AECL will be required. In the case of non-acceptability, alternative proposals must be considered by the governments of Canada and Ontario.

Technical Advisory Committee

The Technical Advisory Committee (TAC) to AECL on the Nuclear Fuel Waste Management Program was established in mid-1979 following recommendations in earlier government reports and suggestions from parts of the scientific community. Its membership was selected entirely from a list of nominees submitted by major scientific and engineering societies in Canada. The purpose of the TAC is to act as an independent review committee advising AECL on the extent and quality of the Nuclear Fuel Waste Management Program. Its responsibilities are to review the content of proposed research projects, to suggest alternatives and additions as deemed appropriate, to review the scientific methods used, to assure that the best available technology is being applied to the program, to review program results and assure that conclusions drawn are valid within the limits that are claimed, and to recommend any specific areas of work for which research should be undertaken, either by existing staff or through research contracts. Its autonomy is assured by the form of appointment to TAC of persons nominated by professional and scientific societies and also by the requirement of reporting in the public domain and by the provision to TAC of full and free access to all aspects of the research program. The Committee is also provided with resources that allow it to obtain additional specialist advice as it deems fit. Its annual reports provide a publicly available documentation of its assessment of the progress and performance within the program. The following sections of this paper reflect TAC's understanding of the program and its progress to date.

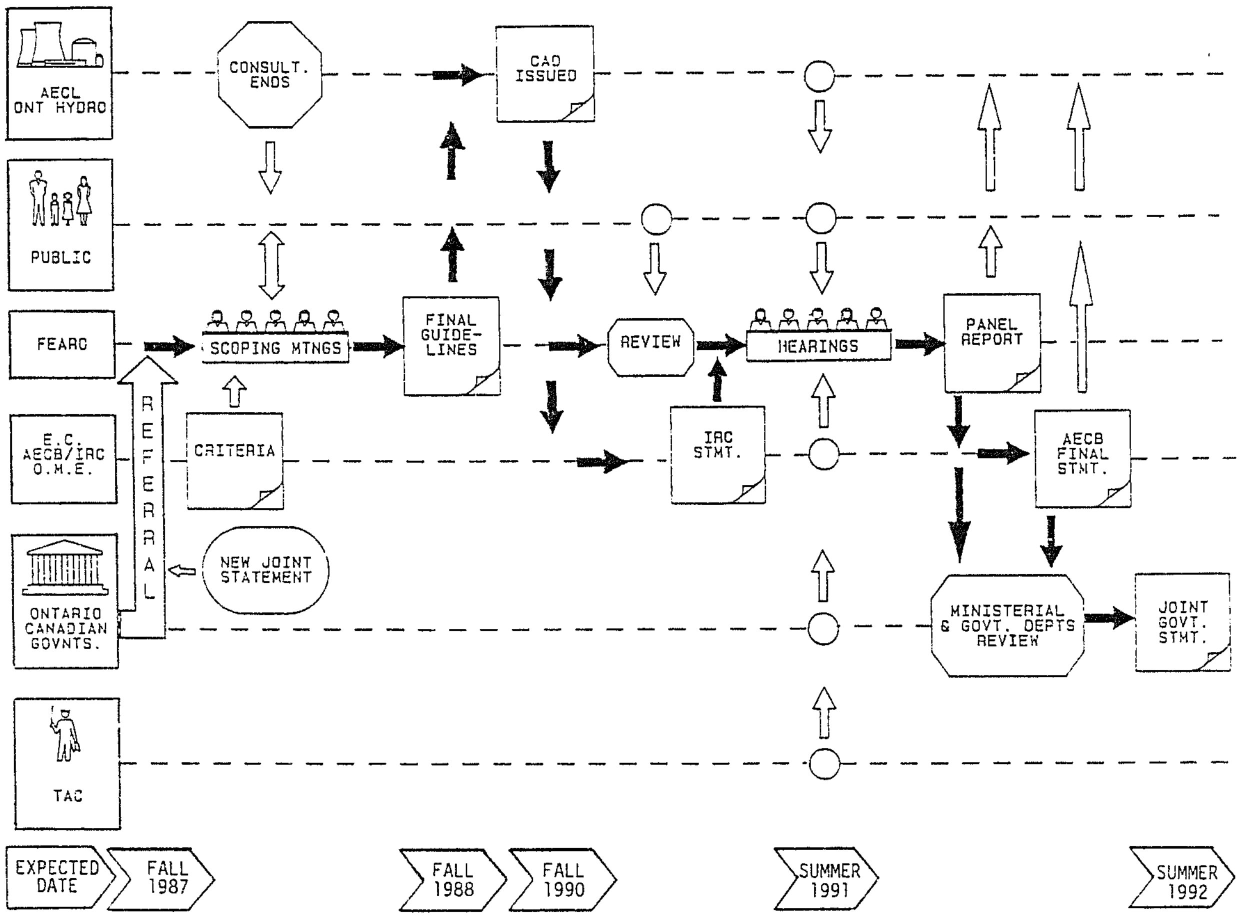

Figure 2: Concept Assessment Review Process and Schedule

PERFORMANCE ASSESSMENT

System Variability Analysis

There are two unique characteristics associated with the permanent disposal of nuclear waste: (a) no precedents exist for such an endeavour, and (b) the very long time-span over which the multi-barrier system must remain effective. To deal with the inherent uncertainties involved in predicting possible future outcomes under this latter condition and reflecting the system complexity, a form of system variability analysis was adopted (Wuschke et al., 1981). In this approach, uncertainties in the data gathered through research and the variation of conditions through space and time are taken into account by assigning a distribution of values to parameters used in modelling or describing the system. The range of distribution is a measure of two entities: (a) our current state of knowledge about a particular parameter based on field and laboratory research, and (b) the inherent degree of variability of a particular parameter. By repeatedly sampling from these distributions of parameter values and subsequently performing simulation studies using these data, a prediction is made of the range of possible effects and their corresponding frequency of occurrence.

The SYVAC Computer Code

This method is applied through a computer program, SYVAC (SYstem Variability Analysis Code) (Wuschke et al., 1981) and is illustrated in Figure 3. System simulations are performed by linking a set of three submodels representing the three major constituents of the disposal system: the vault, the geosphere and the biosphere. Significant processes and conditions within each are characterized by sets of equations. These are, in effect, mathematical statements of the current state of knowledge about the disposal system and the phenomena that influence it.

Initial data input consists of the inventory of radionuclides placed in the vault. The processes of leaching and transporting by groundwater within the vault, together with the reaction of these radionuclides with engineering

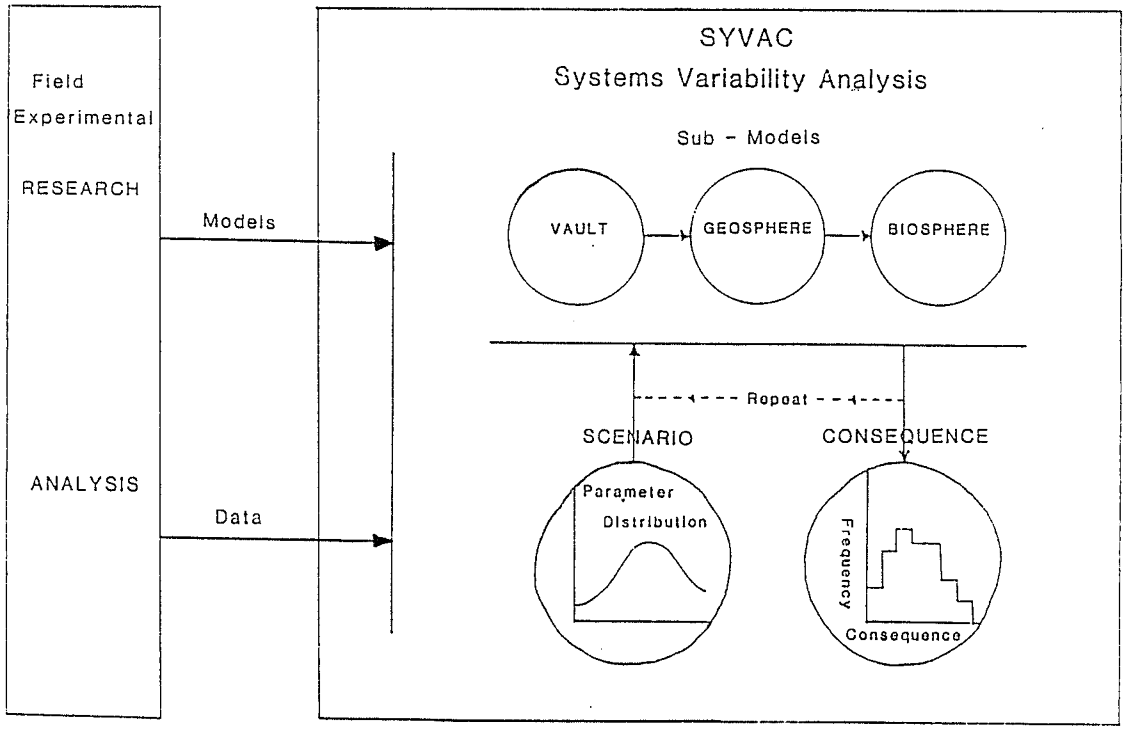

Figure 3. Systems Variability Analysis.

barriers such as buffers and backfill, and the final movement out of the vault are simulated. The output terms (i.e. integrated flux of radionuclides) from the vault serve as the input terms for the geosphere model. Examples of parameters which are taken into account in the estimation of radionuclide movement through geologic media include groundwater velocity, effective path length, chemical retardation factors, etc. In the third and last model, the biosphere, the analysis involves the estimation of radionuclide movement leaving the geosphere and travelling through shallow groundwater, surface water, soil, plants, animals and, finally, to man.

SYVAC treats the three submodels in sequence and produces for a particular scenario (i.e. the situation defined by one particular set of values derived from the random selection of one value from each probability distribution for each parameter), an estimate of the maximum dose to an individual in the most exposed group within a given time after disposal. The maximum dose is termed the "consequence" for that scenario. Estimates of maximum dose from a large number (about one thousand) of such randomly constructed scenarios are plotted to show the frequency of occurrence of any particular consequence. TAC has emphasized the requirement of software quality assurance for such a complex code as SYVAC.

Geosphere Modelling

The geosphere model within SYVAC is largely based on field studies and data from the Whiteshell Underground Research Laboratory (URL) site which is described in the next section. The purpose of the conceptual model, constructed from interpretations of indirect measurements of the real system, is to describe the three-dimensional pattern of groundwater flow within the system and the deep geological features which might control this groundwater system. The model is based on work carried out under a large, extensive program of geoscience research, portions of which are highlighted in the following sections. The need for validation of such a model by application of actual field measurements has been stressed by TAC.

THE GEOSCIENCE RESEARCH PROGRAM

The geoscience program is designed to develop the necessary understanding of processes that may significantly affect the performance of a disposal vault, and to develop a methodology for characterizing a plutonic rock mass to evaluate its suitability for isolating nuclear fuel waste. The program includes five major components:

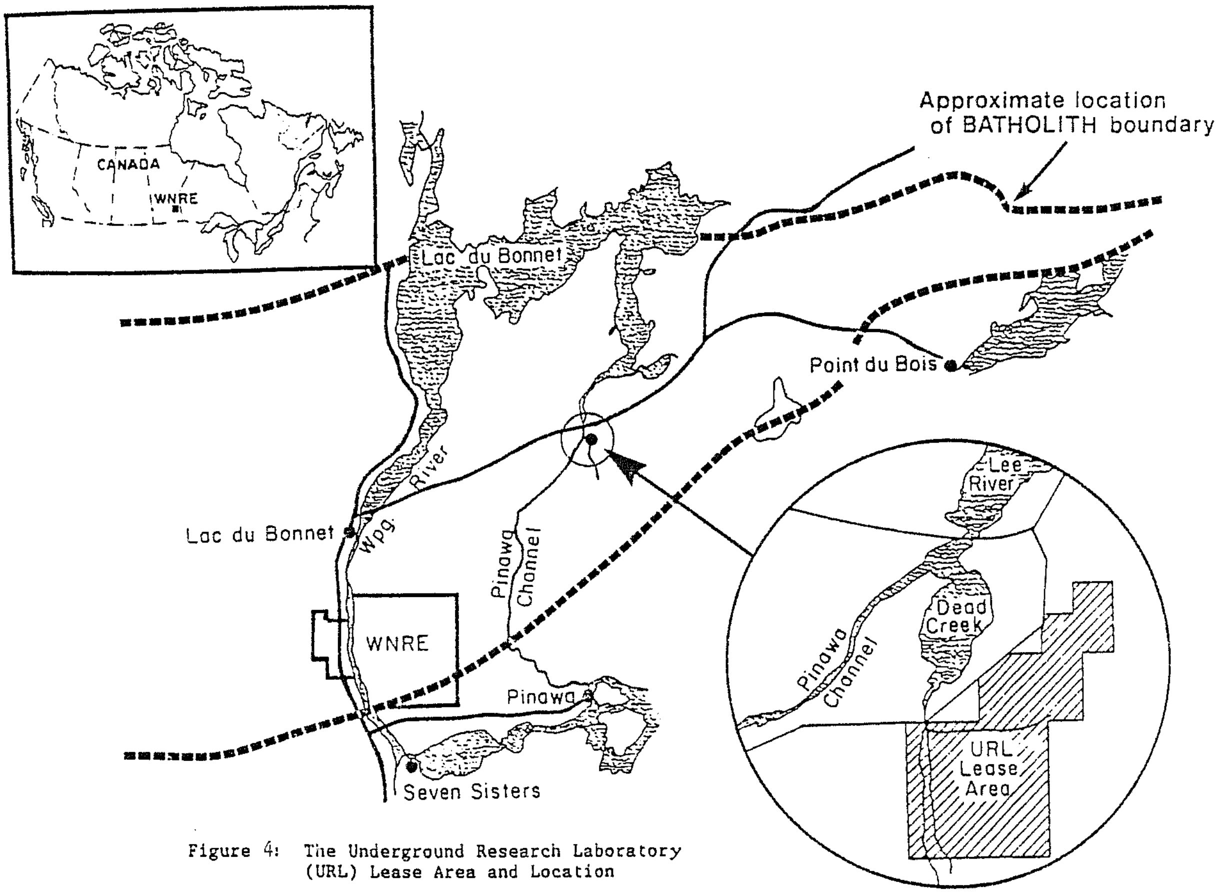

Figure 4: The Underground Research Laboratory (URL) Lease Area and Location

hydrogeology
goephysics
geochemistry
geomechanics and
geology

Some highlights from the hydrogeology and geomechanics programs are described in the rest of the paper.

Research is being carried out in three areas: the granitic Lac du Bonnet Batholith near the Whiteshell Nuclear Research Establishment (WNRE) in Manitoba, the Eye-Dashwa Pluton near Atikokan, Ontario and the East Bull Lake gabbroic intrusive near Massey, Ontario.

Currently the main focus of geoscience field work is a 750 km^2 region on the Lac du Bonnet Batholith in which the WNRE and AECL's Underground Research Laboratory (URL) is situated (See Figure 4).

The Underground Research Laboratory (URL)

The URL is a unique research facility built especially for conducting research to assess the feasibility of deep geologic disposal in the granitic rock of the Canadian Shield. It is situated in an area which had not been previously disturbed and is built below the watertable. It consists of a vertical rectangular shaft (presently at 255 meters deep), a circular ventilation raise and a horizontal access drift at 240 meters. Under a co-operative effort between AECL and U.S. Department of Energy the shaft will be excavated eventually to a depth of 455 meters with another horizontal drift at the 440 meter level (See Figure 5).

Research in hydrogeology, geomechanics and geology are being pursued as part of a complete geoscience program integral with the URL and its construction. For instance, the location of the URL shaft and underground test areas was picked on the basis of underlying geological structure and groundwater flow system inferred from extensive geological and hydrogeological characterization work since 1979. The excavation activities also, rather than being simply a means to an end, are an integral part of the whole geoscience program, providing vital information on the response of both the groundwater flow system and the rock mass. The following sections illustrate the central role that the URL plays in the geoscience program and how it is the focus of varous field and experimental work.

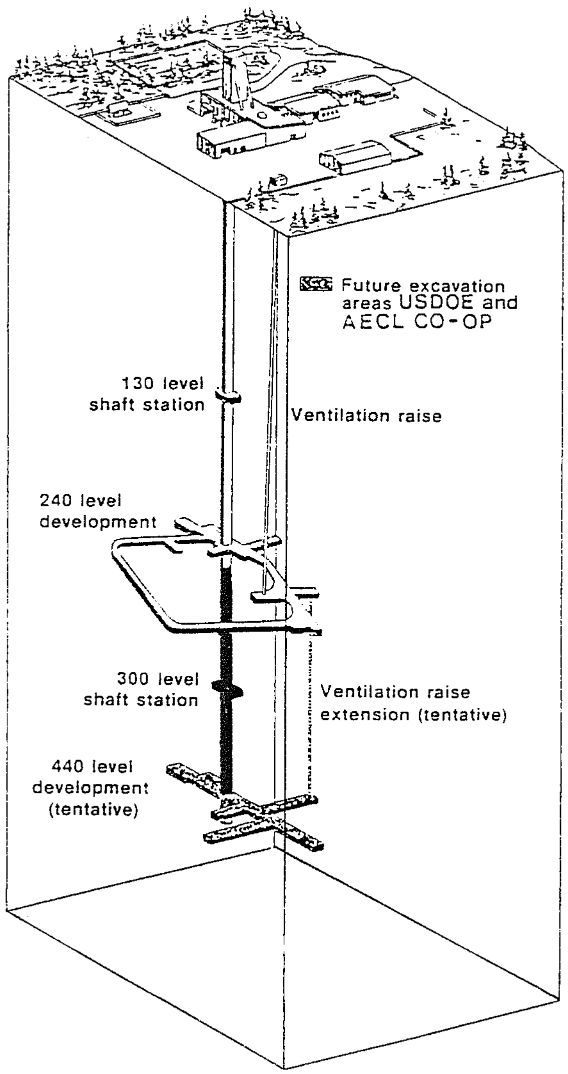

Figure 5. URL Phase 1, Phase 2, and Planned Phase 3 Development.

Hydrogeology

The aim of hydrogeology research is to study those physical and chemical aspects of deep groundwater at a relevant scale and in sufficient detail such that forcasts about its short and long-term flow behaviour on a regional scale in the Canadian Shield can be made with a high degree of confidence. The program began by developing methods to measure physical and chemical properties of groundwater within individual boreholes and fractures. In conjunction with the URL contruction, emphasis now has shifted to the development of methods aimed at the larger site and regional scales.

The present hydrogeology research has developed in four phases (Davison and Guvanasen, 1985). In the first, extensive field investigations was done to determine the geological features of the rock mass that control the groundwater flow and the associated physical, chemical and hydrogeological characteristics. Inferences were then made from these geological features and associated characteristics to develop a conceptual model of the groundwater flow system. Next, a detailed three-dimensional mathematical model of the postualated flow system was developed to predict changes in measurable parameters that would result from natural and artificial perturbations to the rock mass and flow system. Finally, to refine and validate the conceptual and mathmatical models, comparisons are being made between predicted and measured responses. It is hoped that this process will generate a realistic representation of the actual groundwater flow system. Furthermore, it is planned that from the detailed model, a more general and representative model will be derived which may be used in assessment of long term safety of a vault built somewhere in the Canadian Shield.

Field Investigations

The field investigations done in the first phase provided a large amount of subsurface information through an extensive program of borehole drilling, logging, testing, instrumentation and monitoring. In a 4 km^2 area, a large array of boreholes were drilled consisting of 58 water-table wells in unconsolidated overburden, 41 shallow bedrock holes (10 to 60 meters long), and 25 deep boreholes (about 100 m to 1100m in length).

Fractures were characterized in the boreholes using detailed core-logging methods, in-hole television camera equipment, and a variety of standard and innovative borehole geophysical logging techniques (Davison, 1985). Hydraulic conductivity measurements were also made in many of the boreholes at various selected intervals. To obtain hydraulic conductivity values of the portion of the rock mass between boreholes, a number of interference tests were conducted in which water was either injected or withdrawn from one borehole, and the responses were then recorded in an array of isolated intervals in other boreholes. Values of hydraulic conductivities obtained ranged widely from 10^{-12} m/s to 10^{-3} m/s (Davison, 1985).

Conceptual Model

Analysis of information from the entire array of boreholes indicate that three major fracture zones dipping slightly to the southeast in the rock mass control the movement of groundwater in this area (Figure 6). The structure of fracture zone 2 is fairly complex involving a number of off-branching, interconnected zones while zone 3 is relatively uniform and only a few meters thick. The surrounding rock is relatively unfractured except for sets of near vertical fractures which intercept the surface from depths of 100 to 300 meters. Their dominant northeast-southwest orientation is roughly parallel to the direction of the maximum principal stress but vary considerably in the extent of their development.

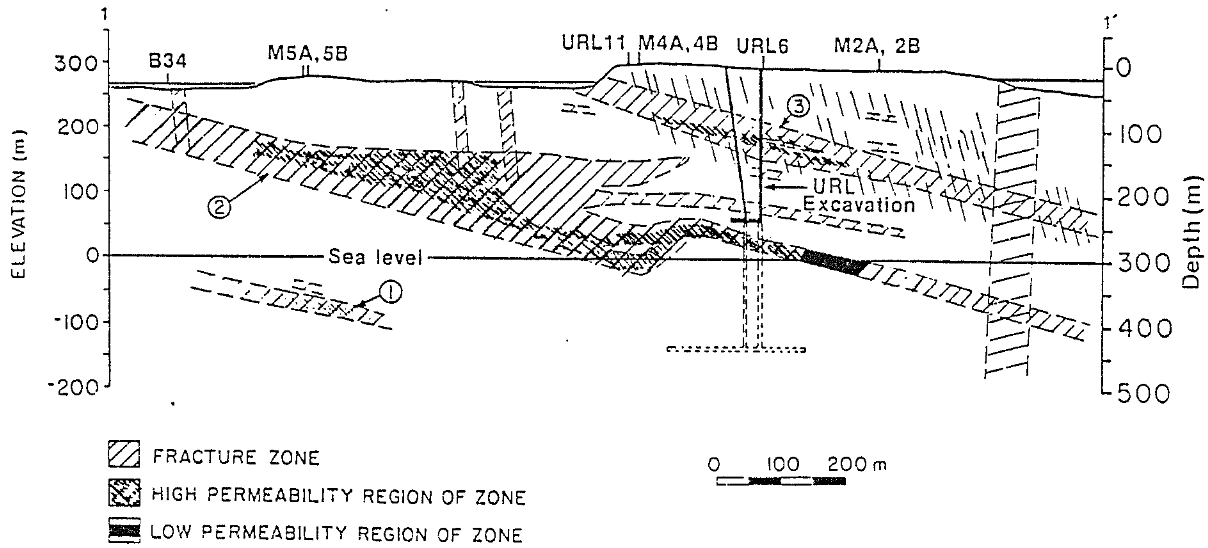

Figure 6: Simplified Geological Cross-Section of URL Site Along 1-1'

The general features of this system are similar to those of other rock masses in the Canadian Shield, that is, several discrete zones of intense fracturing which control the groundwater flow embedded in relatively unfractured rock in which the degree of fracturing decreases with depth.

Mathematical Model

From this conceptualization of the groundwater flow system, a three-dimensional, finite element model was developed and implemented with a computer program MOTIF (Guvanasen, 1985). The large rock mass was treated as an equivalent anistropic porous medium of low permeability and porosity while special planar elements embedded in this mesh represent the discrete fracture zones. The flow within these planes is assumed to be along their axes. Porous medium equations describe the three-dimensional flow within this assembly of blocks and planar elements.

Model Validation

The URL shaft excavation work served to validate the conceptual and mathematical models. Response to the excavation was first predicted for each of 174 packer-isolated monitoring intervals in the network of boreholes surrounding the shaft (Guvanasen, 1985) and was subsequently compared with actual measurements as shaft sinking proceeded.

Figure 7 shows the predicted and measured flow of groundwater into the shaft at various times during its excavation. Inflows are over-predicted by a factor of three (Davison, 1986). This discrepancy may be due to deviations of actual excavation rates from assumed rates, the existence of vertical fractures connecting the shaft with a highly conductive sub-horizontal fracture zone, and the effect of stresses caused by excavation on the local hydraulic conductivity of the rock. The maximum inflow occurred when the upper fracture zone was penetrated as predicted. The first measured inflow occurred somewhat earlier than predicted as vertical fractures, not included in the conceptual model, were penetrated. It should be noted that the discrepencies are very small with respect to the variability of the hydraulic values. The measured values are thus in good agreement with those predicted.

Hydraulic heads measurements are also in close agreement with predictions. Figure 8 shows results from one of the monitoring locations in the upper fracture zones that is typical of others. The sharp drop in head is due to the first inflow into the shaft as the vertical fractures were intercepted at 62 meters. Although the drop occurred a little earlier than predicted, the overall results are in good agreement (Davison, 1986).

Ongoing Hydrogeological Work

Work is now in progress to assess the validity and effectiveness of these methodologies when extended to investigations and predictions on the regional scale. A conceptual model for the entire 750 km^2 Whiteshell Research Area to a depth of at least 1000 meters is being developed. On-going field investigations include additional detailed geological and geophysical surveys; expansion of the

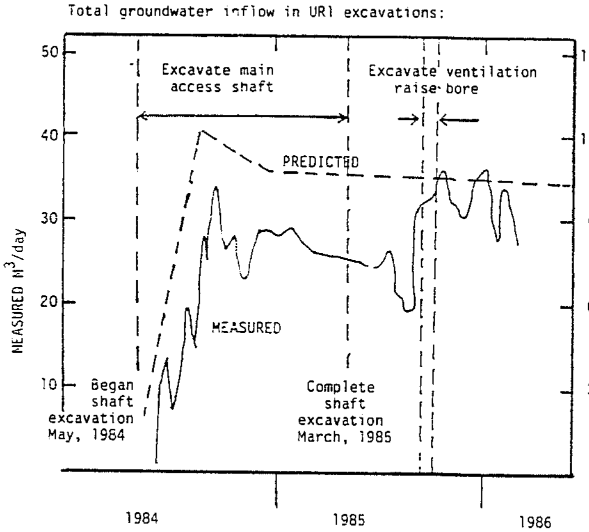

Figure 7. Comparison of Predicted and Measured Groundwater Inflow to the URL Excavation.

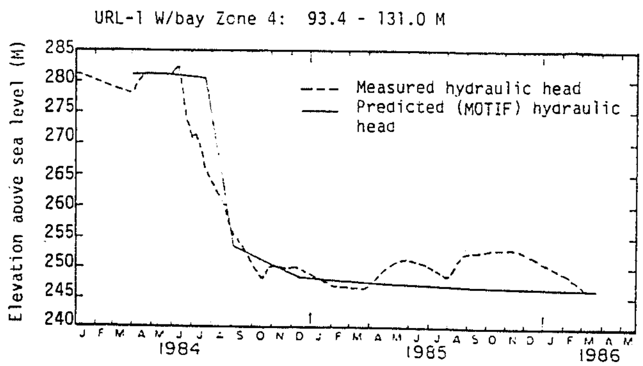

Figure 8. Comparison of Predicted and Measured Hydraulic Head at Borehole URL-1 in Fracture Zone 3.

surface-water hydrology monitoring and meterological monitoring; drilling, logging and instrumenting a network of deep boreholes; and monitoring the groundwater chemistry and hydraulic head fluctuations in the network of boreholes.

Geomechanics

The URL is also a major focus for the geomechanics program. Geomechanics research aims to elucidate the response of a jointed plutonic rock mass to stresses imposed by mechanical and thermal phenomena that would be associated with a disposal vault. In order to find, or check, the values of various parameters needed in the design of a disposal vault, experiments are being carried out both in the laboratory and in the field. They are being

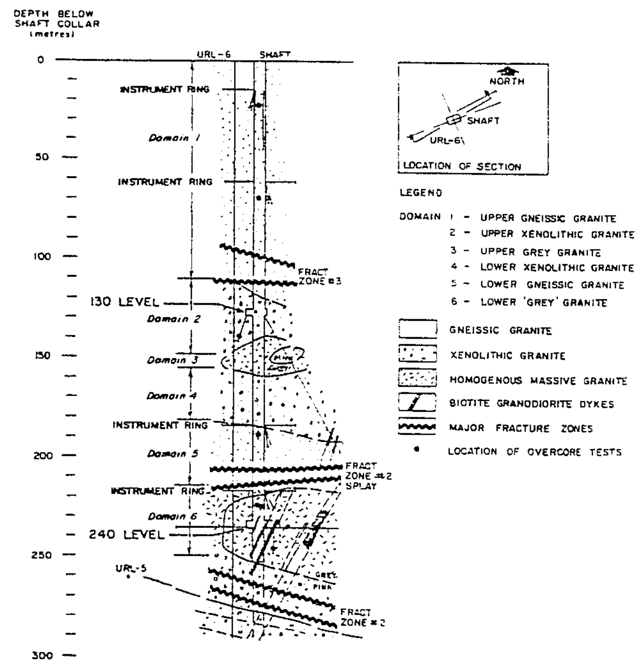

Figure 9. URL Shaft Geology and Location of Initial Overcore Tests for Horizontal Stress Determinations (from Lang et al.,1986).

planned and conducted on three scales: on small specimens obtained by core drilling, on large rock blocks, and by direct observation of large-scale rock mass response to change of stress and/or temperature.

Four arrays of instrumentation are installed at depths of 15m, 62m, 185m and 218m during shaft excavation (Figure 9). A typical array is shown in Figure 10 and includes sonic probe extensometers placed in horizonal boreholes, CSIRO hollow inclusion triaxial strain cells and convergence pins to monitor shaft convergence as excavation proceeds.

Before shaft sinking began, displacement and stress responses were predicted for all instrumented locations using the assumption that the rock mass is a three dimensional, linear elastic continuum (Young's modulus = 40 GPa; Poisson's ratio = 0.26) and that the horizontal principal stresses prior to excavation were normal to the shaft walls (Chan et al., 1985).

Measured values from the extensometer array installed in the rock surrounding the URL shaft at 15 meters are compared with predicted relative displacements in Figure 11. Shown are the displacements at various distances from the shaft along three extensometers (W1, W2, S2), caused by excavating the shaft to a depth 8 meters below the extensometers. The predicted displacements agreed well with those measured along W1 and W2 using a Young's modulus of 32 GPa. Along S2, however, agreement is obtained only after reducing the modulus value to 19 GPa.

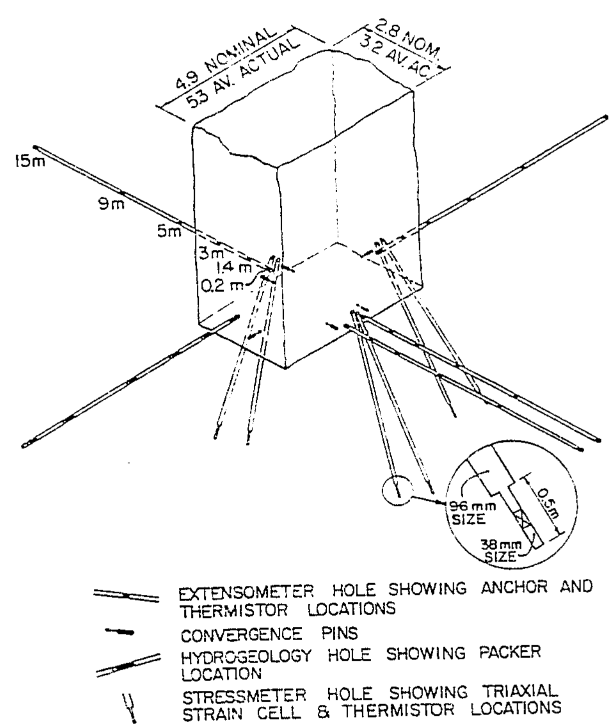

Figure 10. Rock Mass Response Instrument Array (Typical). (from Lang and Thompson, 1986).

The discrepency is likely due to the presence of natural fractures close to the wall where most of the deformation takes place. Below 23m, all calculated displacements underestimate measured values by about 30%. The possibility that this may be due to the thermal expansion of the rock by the warm air in the shaft is being investigated.

These results are typical of a number of such comparisons between predicted and measured displacements, indicating that the relatively unfractured rock can be approximated as a linear elastic continuum with a Young's modulus between 30 GPa and 50 GPa (Chan et al., 1985). However, as might be expected, major natural fractures require detailed treatment.

Ongoing Studies

Major ongoing work within the geoscience program is summarized below:

1. determining the extent and mechanical properties of the damage zone near surfaces of shafts and rooms excavated in a rock mass.

2. application of geological, geophysical and hydrogeological techniques to locate and characterize major natural fractures.

3. development of constitutive relationships for use in mathematical treatments of fractures, using data from both laboratory and field experiments on fractured rock.

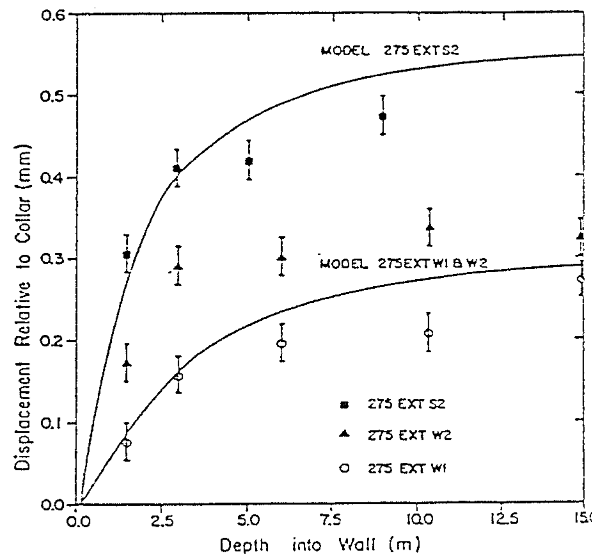

Figure 11. Predicted and Measured Displacements.

4. developing techniques to include mathematical treatments of major fractures in the three-dimensional finite-element models, and then validating the models against field measurements.

CONCLUSIONS

1. An innovative methodology for characterizing the hydrology of a plutonic rock mass has been successfully applied and is being validated in the field. This methodology is now being applied at a regional scale comparable to that required to characterize a candidate disposal site. The process by which detailed in situ measurements are used to develop a conceptual model of the hydrogeology of a site and then idealized into a three-dimensional description is generally valid.

2. Rock deformations predicted using three-dimensional finite-element models have been shown to agree well with deformations measured during excavation of the URL shaft, in portions of the rock mass that do not contain major fractures. Continuing work is focussed on the development of suitable techniques for including fractures in the models.

3. The SYVAC performance assessment methodology, which accounts for uncertainty and variability in system parameters, has been established and is widely accepted as the preferred method of assessing the long-term performance of a disposal system.

4. The Technical Advisory Committee carries out regular periodic reviews of all the activities in the research program, and presents its findings and assessments in a public document.

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The Role of the Geotechnical Engineer in Mine Waste Management

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SUMMARY

Geotechnical engineering plays a major role in the design, construction, and operation of mining projects. The problems that must be addressed range from conventional geotechnical items such as mill foundations, access roads, and pit wall stability to a host of geotechnical problems that are directly related to mine waste management.

The construction in Canada, over the past twenty years, of many, major mining projects has enabled Canadian geotechnical engineers to develop extensive knowledge and experience in the area of mine waste management. As a result, several Canadian geotechnical firms have developed international reputations in this field and now provide these services to the mining industry both in Canada and overseas.

The term "Mine Waste Management" covers a broad spectrum of mine waste items including: tailings storage facilities, mine waste dumps, leach pads, and various waste water problems. This paper will limit itself to a discussion of three mine waste management items that contain a very high geotechnical content, namely: tailings dams; waste dumps; and leach pads.

RÉSUMÉ

Le génie géotechnique joue un rôle majeur dans la conception, la construction et l'opération des projets miniers. Il doit résoudre des problèmes classiques relatifs aux fondations des usines de traitement, aux routes d'accès, à la stabilité des parois excavées, et aussi une foule de problèmes géotechniques directement liés à la gestion des déchets miniers.

Au Canada, la construction de plusieurs complexes miniers majeurs au cours des vingt dernières années a permis aux ingénieurs géotechniciens canadiens de développer un savoir-faire et une vaste expérience. Par conséquence, plusieurs firmes canadiennes ont acquis une réputation internationale dans ce domaine, et elles fournissent leurs services à l'industrie minière tant au Canada qu'à l'extérieur.

Le terme "gestion des déchets miniers" couvre une large variété de problèmes qui comprend: le stockage des rejets, les dépotoirs de déchets, les enclos de lixiviation et divers problèmes d'eaux usées. L'article sera limité à une discussion des trois problèmes de gestion des déchets miniers qui ont une forte composante géotechnique, c'est-à-dire: les barrages de rejets, les dépotoirs et les enclos de lixiviation.

INTRODUCTION

Waste management forms a very important part of every mining operation. To satisfy both safety and environmental requirements, mine wastes must be handled and stored carefully. Consequently, the selection of sites and the design of the necessary facilities become very important items. As the final locations selected for waste storage can have a major impact on mine and mill planning, it is important that all of these activities be integrated and carried forward together.

Geotechnical engineering input is required throughout all phases of mine waste management. The initial input comes with site investigations leading to site selection. Design and construction follow, with the construction of tailings dams and waste dumps being an ongoing operation that continues throughout the life of the mine. On closure, at the end of the life of the mine, reclamation works are required to ensure that stored mine wastes do not pose an environmental hazard.

Two of the three major components selected for review in this paper, tailings dams and waste dumps, are common to most mining operations. The third component, leach pads, is becoming increasingly more common with the use of leaching processes for recovery of gold from low grade ores. All three examples provide a good insight into the extensive geotechnical content involved in mine waste management.

TAILINGS STORAGE FACILITIES

Introduction

Tailings are a waste product of the mining industry. They consist of the ground-up rock that remains after the mineral values have been removed from the ore. The grain size distribution of tailings depends upon the characteristics of the ore and the mill processes used to concentrate and extract the metal values. As shown on Figure 1, a wide range of tailings gradation curves exists for the various mining operations.

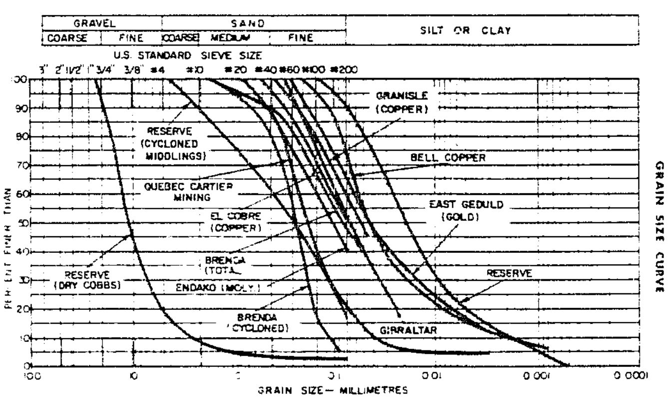

Figure 1: Typical Tailings Grain Size Curves

Tailings normally are transported to the disposal area as a slurry at concentrations of approximately 40% by weight of solids to liquid. The potential pollution hazards associated with storage of the tailings slurry vary with different mining operations, and range from very severe for the radioactive wastes associated with uranium mining, to none for mining processes that merely grind up an inert ore without the addition of toxic chemicals during processing. In between these two extremes are a wide range of conditions that present either short or long-term potential pollution problems. For purposes of this paper, uranium tailings have been excluded as they present unique problems requiring special design, construction, and reclamation problems.

The problems associated with the safe storage of tailings have become greater as rates of production have increased and mining operations have become surrounded by inhabited areas. Figures 2 and 3 illustrate the size of some of the larger tailings dams that are currently under construction. Figure 2 is a section through the Brenda Tailings Dam, which is currently 132 m high and has an ultimate design height of 162 m. Figure 3 is a section through the Lornex Tailings Dam, which is currently 80 m high and may reach an ultimate height of 194 m. Obviously, tailings dams such as these are critically important hydraulic structures, whose safety must be assured both during mining operations and after mining operations have ceased.

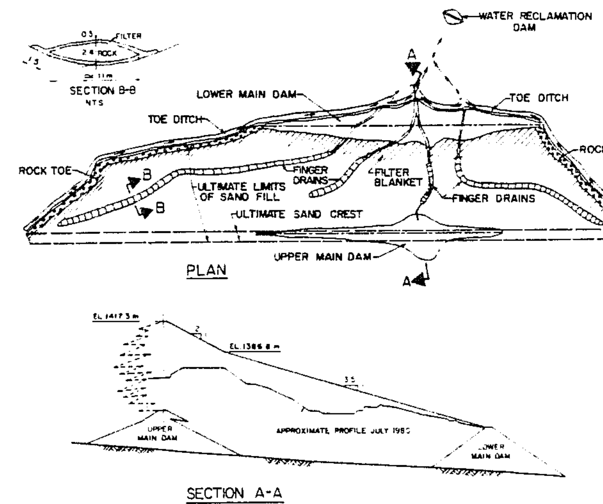

Figure 2: Plan and Section Through Ultimate Brenda Tailings

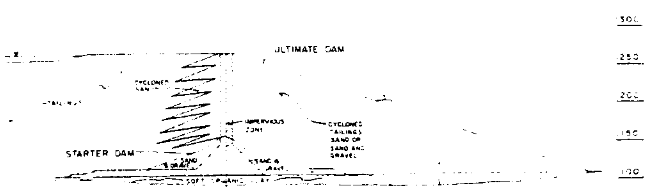

Figure 3: Typical Section Through Ultimate Lornex Tailings Dam

There are two aspects of tailings dam safety that are of public concern. The first is the structural stability of the dam and the possible release, if failure occurred, of a very large volume of water and/or semi-fluid tailings. Such an event would not only cause extensive downstream pollution but would also pose a serious threat to life and property. The second is the possibility of pollution occurring during normal operations by polluted effluent which might escape through or around the tailings dam and into the streams or groundwater of the area. Both these concerns also apply to the tailings facility after the mining operations have ceased.

Geotechnical engineers play an important role in all aspects of tailings dam design, construction, and operation. Their major objective is to provide safe, economical tailings dams that satisfy all regulatory requirements pertaining to safety and environmental impact. In many instances this involves drawing on the wide range of both theoretical and empirical, geotechnical knowledge, available from other engineering fields. However, in other instances unique problems arise which require innovative solutions from the geotechnical engineer. The following sections of the paper briefly cover the wide field of tailings dam design, construction, and operation and attempt to illustrate the wide range of geotechnical items which must be addressed.

Tailings Dam Design

Introduction - Current good engineering practice for the design of tailings dams utilizes the wealth of knowledge and experience available from conventional water storage dam designs. This is a logical development as large tailings dams do constitute major hydraulic structures. The downstream risks presented by tailings dams vary considerably from one mining operation to another. For some operations, particularly in very arid areas, the material stored behind the tailings dam comprises mainly solid wastes, with very little free water. Other operations may store very large quantities of free water as well as solid wastes. Obviously, those tailings dams that store large volumes of water, in addition to tailings, pose the greatest potential downstream risks. However, even those tailings dams that normally store only small amounts of water can pose a threat to downstream life and property under certain conditions such as: unusually heavy rainfall or snow runoff, causing the pond to fill and overtop the dam resulting in a washout and failure; or a large earthquake which exerts inertial forces on the dam while at the same time causing shear strengths to decrease, with the net result being possible instability and failure of the dam. Consequently, the writers believe that all major tailings dams should be designed, utilizing the knowledge and experience developed for conventional water storage dams, suitably modified of course, to satisfy the special requirements of the mining industry.

Types of Tailings Storage Sites - The types of sites that may be developed for the storage of tailings can be classified into five broad categories, as follows:

1. Cross Valley Impoundments
2. Dyked Impoundments
3. Hillside Impoundments
4. Incised Impoundments
5. Combinations of one or more of the above

These are illustrated on Figure 4.

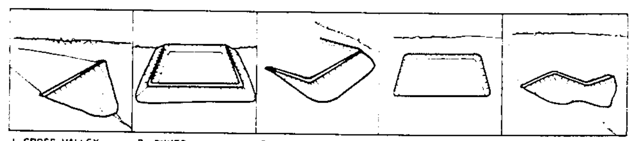

Figure 4: Types of Tailings Storage Sites

Cross Valley Impoundments - The cross valley impoundments are the most common type of tailings facility in mountainous terrain. Their major advantage is that they can store large volumes of tailings by the construction of a single dam across the downstream end of the valley. Their major disadvantage is that facilities must be constructed to divert, store, and/or discharge all of the water contributing to the watershed, upstream of the dam. If the valley has a large watershed, handling the runoff can become a very big problem.

Dyked Impoundments - In flat or gently sloping terrain, impoundments dyked on all four sides are often used. This type of impoundment offers the important advantage of having to handle only that water which falls directly on the enclosed impoundment. However, the costs of building this type of tailings impoundment are often very high as dykes are required on all four sides of the tailings pond.

Hillside Impoundments - Hillside impoundments offer the advantage of requiring limited diversion facilities to handle surface runoff while requiring far less dyking volumes than the dyked impoundment.

Incised Impoundments - Incised impoundments are usually constructed on fairly level terrain and are limited to relatively small tailings storage facilities. Two advantages of incised impoundments are: they have limited runoff to handle; and they pose simpler reclamation problems. A disadvantage is the cost of excavating a portion of the storage pond.

Combined Impoundments - One type of combined impoundment sometimes used is the "cross-valley-hillside". This combination impoundment offers the same advantages and disadvantages as its two components.

Methods of Tailings Dam Construction - As noted previously, safe and economical tailings dams can be built by applying the engineering knowledge and experience currently available from conventional water storage dam designs, suitably modified to satisfy the special requirements of the mining industry. Tailings dams are currently being designed and constructed, using sound engineering principles, to produce safe dams at costs that are only slightly greater than those for the older, and in many instances, unsafe structures.

Upstream Construction Using Tailings - In the past, practically all tailings dams were constructed by some variation of the upstream method of construction. The original upstream method normally involved construction of a low earth "starter" dyke, 3 to 6 metres in height. This dyke was usually constructed from locally available borrow materials and was seldom subject to engineering design. The tailings were discharged by spigotting off the top of the starter dyke. When the initial pond was nearly filled, the dyke was raised using borrow material from the dried surface of the previously deposited tailings, and the cycle was repeated. As the height of such a dam increases, each successive dyke moves further upstream, and is underlain by the soft, previously deposited tailings.

Upstream methods of construction have been used successfully in dry, arid climates, where evaporation losses are high and a minimum of water is stored in the pond. Under these conditions the phreatic line through the tailings dam is low, some chemical cementing of the dried-out tailings occurs, and provided sufficiently flat outer slopes are used, significantly high tailings dams can be successfully constructed under static loading conditions. The major risks with this type of construction are associated with failure to maintain a very small volume of water in the pond and subsequent raising of the phreatic line. Under these latter conditions failure may occur under static loading and likely will occur under large earthquake loading, owing to the development of high pore pressures and possible liquefaction.

Sub-aerial methods of tailings deposition are sometimes used with the upstream method of dam construction. Sub-aerial tailings disposal is a method of depositing tailings in thin layers on a gently sloping beach, in order to achieve a denser, less permeable, higher strength deposit through settling, draining, and air-drying. Sub-aerial deposition, combined with upstream construction, is a more appropriate technology for warm, arid regions than for wet or cold climatic conditions.

The major shortcoming of all methods of upstream construction is the lack of engineering control over the material which underlies the tailings dam as it is raised in the upstream direction. Figure 5 presents a comparison between a conventional water storage dam and a tailings dam built using the upstream method of construction.

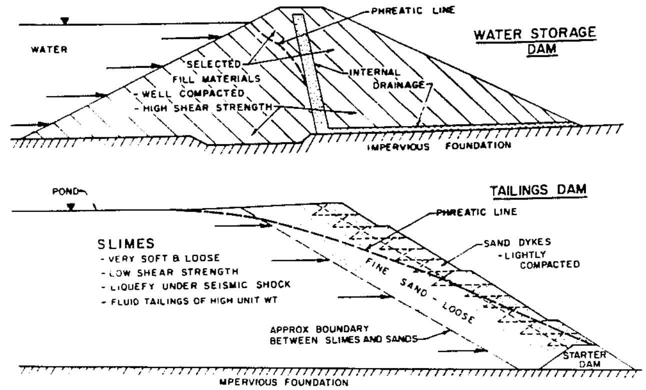

Figure 5: Comparison Between Conventional Water Storage Dam and Tailings Dam Built Using an Upstream Method of Construction

Downstream Construction Using Tailings - Current, good engineering practice is to use downstream methods of tailings dam construction for all major tailings dams. In areas of high seismic risk, where failure of the tailings dam poses a threat to life and property, downstream methods of tailings dam construction should be used for all structures regardless of their height. The downstream method of tailings dam construction evolved from a blending of the engineering knowledge and experience available in the field of water storage dams, with the knowledge of the mining operators responsible for construction and operation of tailings dams. The downstream method of tailings dam construction involves constructing a dam in a downstream direction from the initial starter dam, using the coarser fraction of the tailings. Consequently, as the dam is raised, it can be constructed over a carefully prepared foundation base rather than over previously deposited slimes, as is the case for the upstream method. Most downstream methods of construction process the tailings with cyclones (hydrocylones) to separate the sand sizes from the slimes. The sands are then used in dam construction and the slimes are deposited in the tailings pond.

Figure 6 presents a comparison between a conventional water storage dam and a tailings

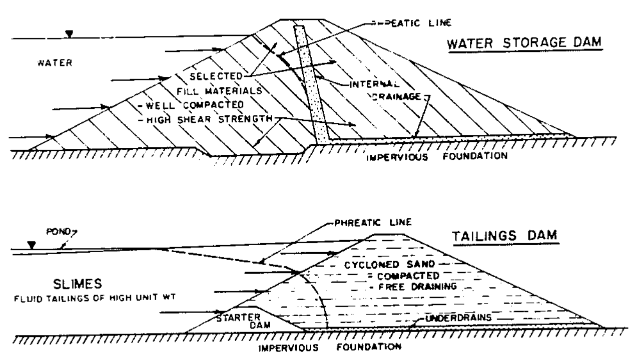

Figure 6: Comparison Between Typical Water Storage Dam and Tailings Dam Built Using One of the Downstream Methods of Construction

dam built using the downstream method of construction. As might be expected, the downstream method of tailings dam construction permits far better control of seepage flows and pressures than does the upstream method.

Conventional Water Storage Dam - In some instances it may be desirable to use a conventional water storage dam to retain the tailings. The dam is normally built in stages in the downstream direction. This may be the case when the tailings themselves are unsuitable for dam construction. This condition can arise if the tailings are very finely ground and contain very little sand sizes, or where the tailings sand contains environmentally unacceptable ingredients such as pyrite, which will oxidize and produce acid effluent. In some instances the availability of mine waste, suitable for dam construction, makes it economically feasible to construct the tailings dam as a conventional water storage dam, even though the tailings themselves might be suitable for dam construction. Figure 7 presents a section through such a dam. The fine tailings provide the upstream impervious zone.

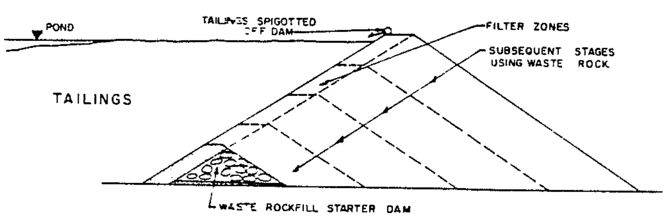

Figure 7: Semi-pervious Tailings Dam Constructed Using Waste Rock

Closed Circuit Tailings Ponds - Dam safety and environmental pollution control regulations can have a major effect on the seepage control facilities required for either upstream or downstream tailings dams. In those instances where a closed circuit tailings pond is required (no discharge of effluent is permitted downstream of the dam), foundation cutoffs and foundation drainage wells may be necessary under either the main tailings dam or a downstream seepage recovery dam, to prevent surface, embankment, foundation, or abutment seepage from passing downstream of the tailings storage facilities. Figure 8 schematically illustrates a closed circuit tailings pond.

Basic Design Requirements - A detailed description of the many aspects of tailings dam design that must be considered by the designer are covered in the engineering literature (Klohn 1982 and ICOLD 1982). A brief summary of some of the more important aspects of tailings dam design which must be addressed follows.

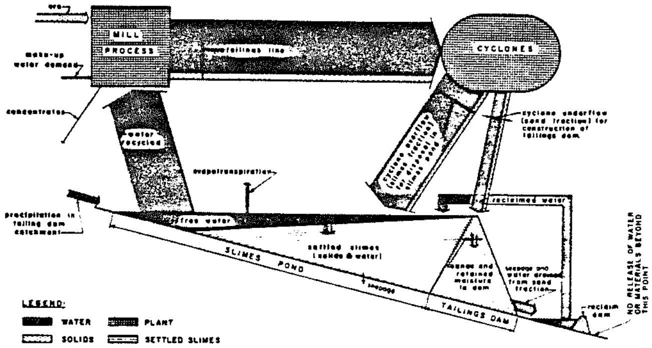

Figure 8: Water Balance for Typical 'Closed Circuit' Tailings Dam

- Site Investigations and Laboratory Testing - The site investigations and laboratory testing programs must be broad in scope and should define the general geology, the soil and rock stratigraphy, and the physical, shear strength, compressibility, and permeability characteristics of both the in situ materials and the mine waste or other borrow materials planned for use in construction of the tailings dam. The steps that should be followed in carrying out detailed site investigations for tailings dams are outlined in the previously noted references.

- Stability of Abutments and Foundations - Abutment and foundation stability assessments for a tailings dam are carried out in the same manner as those made for a conventional water storage dam. The problems are site specific and all investigations, analyses, and designs must be carried out to satisfy any special conditions or requirements of the particular site.

- Stability of Embankment - The stability of the embankment is controlled by its shape (i.e. height, slopes, berm and crest widths, etc.), the strength properties of the materials that make up the embankment, and the location of the phreatic line. The shape of the embankment is easily controlled and monitored. The physical properties of the embankment will depend on the design selected and the method of construction used and usually can be fairly well-defined. The location of the phreatic line can be readily measured with piezometers installed in the embankment. However, the location of the phreatic line can vary widely as it depends on such items as: width of the tailings beach, permeability of the embankment and effectiveness of any internal drainage. A high phreatic line may not only significantly reduce the stability of the dam, but might also lead to concentrated seepage flows and piping. Control of the phreatic line is therefore an important consideration in maintaining an adequate factor of safety for the stability of the embankment.

Seepage Losses Through Abutments and Foundations - Predicting seepage losses through the abutments, the embankment, and the foundations is very important for assessing both the physical stability of the tailings dam and the possible introduction of polluted effluent into the ground and surface waters downstream of the tailings dam. This problem is evaluated using the procedures conventionally applied to the seepage analyses of water storage dams.

Water Balance of the Tailings Pond - The water balance of a tailings pond is the summation of all the water that enters the pond less all the water that is removed or lost through seepage. The sources of water entering the pond are:

- the tailings transportation water
- runoff from the watershed that contributes to the tailings pond.

The water losses from the pond are:

- water reclaimed to the mill
- evapotranspiration losses
- seepage losses
- water stored in the voids in the tailings
- any discharges from the pond

To make a water balance calculation, the designer must have accurate data concerning: mill production rates, concentration of slurried tailings, bulk density of deposited tailings, reservoir capacity volumes, mill reclaim water volumes, evapotranspiration losses, seepage losses, and hydrology of the watershed for the tailings pond.

A review of the above items indicates that most are well-defined and can be accurately determined from year to year. However, runoff from the contributing watershed into the tailings pond is a major item that can vary widely from year to year. Excess water entering the pond must either be stored, discharged downstream, or handled by combined storage and discharge. If the tailings effluent is considered toxic and environmentally unacceptable, no discharge downstream would be allowed. In the latter case all the runoff must be stored in the tailings pond. Depending on the size of the runoff and the type of tailings dam, storing the water either may be quite acceptable or may cause serious concerns about the safety of the embankment. Concern for the safety of the dam would arise if the volume of runoff caused the tailings beach to be drowned and placed the free water in the pond against the upstream face of a sand dam. This is illustrated in Figure 9. Under these conditions a high phreatic line could develop, leading to large seepage losses and raising the possibility of piping developing through the sand dam. This would be a serious condition requiring immediate action to prevent failure. An even more serious condition would develop if the runoff surcharge was so large that the dam was overtopped, as failure would be inevitable.

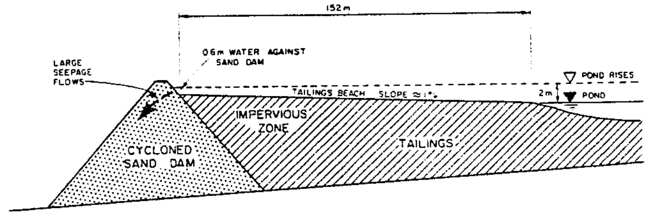

Figure 9: Sketch Illustrating Flooding of Tailings Beach by Surcharge in Tailings Pond

From the above discussion it is apparent that accurate knowledge must be developed concerning the water balance for a tailings pond, and particularly for the hydrology of the contributing watershed. Even for the more favourable case, where water may be discharged downstream, the volume of water to be stored and/or discharged must be known to permit sizing of the spillway facilities. Current practice on most large tailings dams is to use the PMF as the design flood. This procedure is recommended by ICOLD in their Manual on Tailings Dams and Dumps. (ICOLD, 1982).

- Decant and Spillway Structures - As noted in the previous section, spillway structures for major tailings dams should be designed to handle the PMF. In most cases this is done by combining a spillway discharge with pond storage to reduce the size of spillway required. However, it must be remembered that if a tailings beach and sand dam are being used, the pond surcharge must not flood out the beach and place free water against the sand dam.

Decant towers and pipelines passing through the dam must be carefully designed to prevent failure and possible loss of tailings and even piping through the dam. These structures are particularly vulnerable in regions of high seismicity.

- Seismicity of the Region - Perhaps the single most significant factor affecting the safety of tailings dams is seismic loading. The earthquake forces may affect the dam by the addition of inertial loads, the development of high pore water pressures, or a combination of both. In the extreme, liquefaction of loose, saturated tailings could occur with catastrophic results.

Tailings dams located in seismic areas must be designed to resist the seismic forces. Current practice is to design the tailings dam to resist the Maximum Credible Earthquake (MCE) for the region. (Klohn 1982, ICOLD 1982). Procedures for determining the MCE are outlined in the engineering literature and will not be discussed herein. Current state-of-the-art procedures should be used to analyze the stability of the tailings dams under seismic loading. (Klohn 1984). These normally involve determining the response of the dam to the MCE and estimating both its stability at the conclusion of the earthquake and the amount of movement that occurs during the earthquake.

- Instrumentation - Field instrumentation and subsequent monitoring are important aspects of tailings dam design just as they are for conventional water storage dams. They begin with the original site investigations and continue throughout the life of the structure, including the construction, operation, and reclamation stages. The instrumentation should include such items as: piezometers, settlement gauges, inclinometers, seepage quantity measurements, and water quality measurements.

The data obtained from the instrumentation and monitoring programs determine the actual performance of the tailings dam and allow this to be compared with the predicted performance. If the actual performance falls short of that required, remedial measures can be taken to correct the situation. Another advantage offered by a good performance monitoring system is that it usually will provide the operators with advance warning of any potentially hazardous, developing problems.

Safety of Tailings Dams

Introduction - There are two basic items to be considered when addressing the problem of "safety" for tailings dams. The first item is the physical safety of the embankment against failure by such mechanisms as: sliding, slumping, overtopping, piping, etc. The second item relates to the safe containment of any toxic materials that might be stored in the tailings pond. Obviously, the physical integrity of the embankment could be perfectly satisfactory, but toxic materials could be lost from the tailings pond. Such conditions could exist both during operation and after closure. Consequently, when dealing with tailings dams the words "dam safety" need careful definition.

The writers suggest that the general topic of tailings dam safety be divided into two separate items as follows:

1. Dam Safety . covers the physical safety of the embankment and related structures.

2. Environmental Safety. . covers all aspects of storing toxic materials without creating unacceptable levels of environmental damage.

The first item "Dam Safety" is the item that comes to most engineers' minds when tailings dam safety is raised. The second item, however, is becoming increasingly important as society becomes more and more aware of the problems associated with environmental damage. In many cases the second item "Environmental Safety" has an important bearing on the design, construction, operation, and abandonment phases of a tailings dam.

The physical safety of a tailings dam depends on many factors, the more important of which were discussed in the previous section of this paper. Reviewing the physical safety of a tailings dam is a fairly straightforward exercise provided all of the required basic data are available. For new designs this is usually the case with the data obtained from the installed instrumentation providing the required check on whether or not the dam is performing as designed. For existing tailings dams the task may be much more difficult if records are missing concerning the original basis of design, construction methods, or performance monitoring data. Of particular importance is the performance monitoring data as it provides information on actual pore pressures, settlements, seepages, water quality, and lateral movements. Where such records are missing, extensive field investigations must be carried out to obtain all of these data so that the required check analyses can be made. However, even extensive field investigations often cannot provide the invaluable data that would be obtained from good monitoring records extending back to the beginning of construction.

The environmental safety of a tailings dam poses a more complex problem. The tailings dam may be safe physically, but may be losing pollutants by seepage through the base and sides of the pond, through the dam abutments, through the dam foundations, or through the embankment itself. Moreover, seepage losses which are not considered to pollute the surface and groundwater initially, may become pollutants with the passage of time. This can occur if acid producing materials such as pyrite occur within the tailings. Acid production can change the pH of the tailings effluent causing heavy metals to go into solution and to be carried downstream by seepage flows. Similarly, where pyrite occurs in the tailings sand used to build the dam, rainwater seeping through the sand can produce an acid effluent which can pollute downstream stream flows. Obviously, the entire area of environmental safety poses a special problem that is site specific as it depends not only on the imperviousness of the pond and dam but also on the physical properties of the tailings and the re-agents used in the milling process. Assessing the environmental safety of a given tailings facility requires knowledge of physiochemical and chemical reactions as well as seepage flows and groundwater movements.

Phases of a Tailings Dam - Tailings dams can be considered to pass through three distinct phases as follows:

- operating phase
- transition or reclamation phase
- long-term phase

Operating Phase - During the operating life of the tailings dam, the mine operators and/or their designers monitor and control the tailings facility to ensure that it meets the requirements of the involved regulatory agencies. Consequently, dam and environmental safety requirements are usually satisfied during this stage.

Transition or Reclamation Phase - When mining operations cease the tailings pond passes through a reclamation phase. This phase normally involves draining the pond and diverting surface water, sealing the surface of the tailings pond, providing vegetative cover, and providing whatever permanent spillway facilities that might be required to handle surface runoff. The reclamation procedures follow a plan that has been previously submitted for approval to the regulatory agencies. Over a period of time, during the reclamation phase, the characteristics of both the water associated with the tailings pond and the tailings themselves change. Eventually, an equilibrium condition will be reached between the surface flows that enter the pond area and the seepage or discharge of contaminants from the pond. The end of this reclamation stage is normally considered reached when the toxicity of any contaminants released approaches the natural background level or otherwise environmentally acceptable levels.

Long-Term Phase - The long-term phase is considered to start at the end of the reclamation stage. The duration of the long-term phase depends on the duration of the chemical reactions that occur within the tailings pond. These reactions are known to last for long periods, in some cases several hundreds of years. In preparing the tailings facility for the long-term stage this fact should be kept in mind and the final reclamation procedures should be designed accordingly. This usually means incorporating design features to protect against such items as:

- water pressure and floods
- water erosion
- wind erosion
- weathering of construction materials
- earthquakes

The discussion of long-term environmental problems that might be associated with the storage of tailings is beyond the scope of this paper. The issue is raised to remind the reader that any potential long-term environmental problems should be assessed during the design stage of the tailings storage facility.

MINE WASTE DUMPS

Introduction

Mine waste dumps are the disposal areas for the unmineralized or uneconomic rock or soil which is excavated during the mining process to allow access to the ore, the rock containing economic grades of mineralization. Waste rock and overburden dumps are of most concern in surface mining applications, in which high ratios of waste to ore removed result in large tonnages of waste materials. In large open-pit, base metal or coal mines, waste quantities of up to several hundreds of thousands of tonnes per day may be generated. Economic considerations usually dictate dump geometry, as the waste materials must be dumped within the shortest practical haul distance. The role of the geotechnical engineer in mine waste dump planning is to determine dump designs and operating practices that will satisfy both safety and environmental requirements consistent with acceptable costs.

Mine Waste Characteristics

The properties of mine waste materials are highly variable and must be determined in the dump design process. Investigations for new mine dumps must be carried out to determine the anticipated characteristics of the waste. Rock strength may be evaluated by testing of drill core by Point Load strength index tests or Unconfined Compression tests. Strengths of greater than 50 MPa indicate relatively hard and durable rock. Susceptibility to mechanical breakdown can be assessed by the Los Angeles Abrasion Test. A Los Angeles Abrasion index of more than 40% to 50% indicates a friable rock which will break down during mining and handling (Claridge et al, 1986). Resistance to physiochemical degradation can be assessed by slake durability tests, with a slake durability index of less than about 90% indicating a rock which may break down on exposure. Logging of the spacing of discontinuities of waste rock will indicate the anticipated grain size distribution of the waste.

Coal mine wastes consist of sedimentary rocks ranging from competent sandstones and hard siltstones to weak, fine-grained shales, mudstones, and coaly fines. The latter, weaker rocks will usually result in fine-grained wastes.

Metal mine wastes are often relatively hard, metamorphic and igneous rocks, although there are many examples of highly fractured, altered or weathered host rocks which produce fine-grained wastes.

Asbestos orebodies usually occur in metamorphosed rocks, particularly argillite, which breaks down on mining to predominantly sand and gravel sizes.

Overburden waste, the native soil cover overlying bedrock, may be common to any type of mineral deposit. Overburden waste is often cohesive and may have to be handled separately from waste rock.

Waste Dump Configurations

The four common configurations for mine waste dumps may be categorized as follows:

1. Wrap-around Dump
2. Upslope Dump
3. Single Lift Dump (Free Dump)
4. Free Dump with Toe Berm

The dump configurations are discussed below and are illustrated in Figure 10.

Wrap-Around Dump - Wrap-around dumps can be conveniently constructed when mining operations are proceeding to progressively lower elevations. Ideally, each lift is ceased at or before the point of marginal dump stability, and each successively lower lift

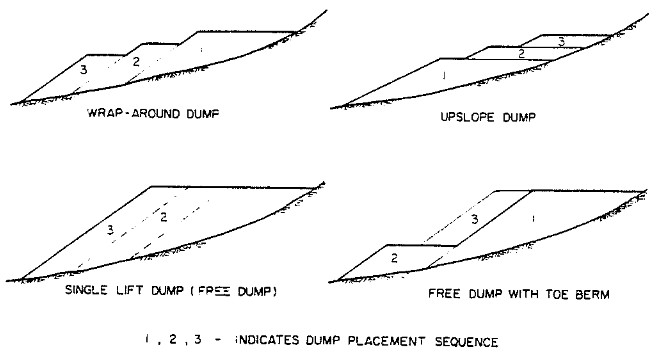

Figure 10: Typical Waste Dump Configurations

wraps around as a buttress to the adjacent upper lift. The advantages of this configuration are that adequate stability can be maintained throughout the dump sequence and that the individual benches can be re-sloped later for reclamation.

The disadvantage of wrap-around dumps is that, should mine operations cease prematurely for economic reasons, high dump slopes may be left without wrap-around buttresses.

Upslope Dump - Construction of dumps in an upslope sequence is most desirable. This method is carried out where mining operations are proceeding upslope or where dumps are constructed progressively higher atop relatively flat topography. The stability of each individual bench and of the overall dump is certain at all times. Mine operations can cease at any time and leave a suitable configuration for dump reclamation and abandonment.

Single Lift Dump (Free Dump) - Single lift dumps, or "free dumps" are commonly used where mining operations are carried out on a mountaintop and waste cannot be economically hauled to lower elevations. As dumping proceeds, the dump face becomes progressively higher, with increasing hazard of instability. In several large Canadian Rocky Mountain coal mines, free dumps are being operated with slope heights of 200 to 300 m, with common occurrences of slope failure. Reclamation of such dumps is extremely difficult, as these high dumps cannot be economically resloped to a long-term, stable angle to support vegetation.

Free Dump with Toe Berm - Toe berm dumps may be designed either to accommodate waste from pits operating at both high and low levels or may be specifically constructed to provide toe support for a high, free-dumped face which might otherwise have inadequate stability. Toe support can usually be provided by a relatively small waste volume. Such configuration is superior to a free dump in stability aspects but, like a free dump, cannot be economically reclaimed.

Water Handling

Design of mine waste dumps requires consideration of water handling, both to avoid erosion or destabilizing of the dump and to avoid impacting downstream water courses.

Normally, runoff diversion ditches are constructed to intercept flow which would enter the waste dump from upslope.

In many instances in mountainous regions, the most economic or the only available locations for waste rock disposal are in stream valleys. Provision must be made for conveyance of streams either through or around waste dumps. Hydrological investigations and analyses must be carried out to determine the design streamflow. A 200-year return period flow is often considered acceptable for design, along with consideration of the consequences of a larger event such as a PMF.

The two main options for conveyance of streamflow are:

a) Diversion channels
b) Rock drains

Diversion Channels often can be constructed to provide satisfactory diversion around waste dumps during operation, provided they can be made accessible for inspection and maintenance. However, diversions usually cannot be considered acceptable solutions for long term closure. Diversions may have to be backed up with rock lined spillways constructed over the dump surface after closure of the mine.

Rock Drains for conveyance of streamflow beneath waste dumps have become increasingly accepted as the preferred design for valley fill waste dumps (Lighthall and Sellars, 1987). Rock drains can usually be constructed of the waste rock itself. The most economical technique is to dump select, coarse waste rock from a minimum height of about 20 m so that natural segregation places coarse particles in the valley bottom. Where the gradation of waste rock is not sufficiently coarse, the rock drain may have to be constructed by special placement of rock, using material selected from the waste rock, or separately quarried.

Rock drains should be constructed of durable, hard rock. They can be constructed with enlarged entrance capacity at the upstream end to allow for some blocking by sedimentation. They are expected to provide improved long-term performance over diversions. However, to date there is limited experience with long term performance of rock drains so their use as permanent, long-term solutions is presently unacceptable, except perhaps for very minor catchment areas.

Stability Considerations

Stability Investigations - Foundation investigations for waste dumps usually consist of geologic mapping, test pitting, and, if required, geotechnical drilling. Sufficient geologic work must be carried out not only to determine geotechnical foundation properties but also to ensure the dump is not covering economic mineralization.

Stability analyses must be carried out for dump design. Normally, a minimum factor of safety of 1.3 for deep-seated movements is desirable for operating dumps. Analyses may point out areas where stripping of weak soils in dump toe areas should be carried out.

Locations of groundwater discharge should be noted and provision made for placement of a coarse rock drainage layer in these locations.

In northern locations, areas of permafrost must be noted and, if possible, avoided for placement of waste. Covering with waste dumps will generate high pore pressures as permafrost thaws, resulting in creep movements which may continue for many years. Such instabilities may involve entire dump masses.

Modes of Waste Dump Instability - The factor of safety of failure surfaces at shallow depths below a slope at angle of repose will always be near unity. Hence, ongoing shallow sliding of active dump faces is to be expected.

Slip surfaces involving failure through or along the dump foundation must be analyzed. Consideration must be given to the potential for pore pressure buildup. Pore pressure may build up either through poor drainage in the lower levels of fine-grained waste material or where significant depths of compressible soil exist in which pore pressures generated by waste dump loading dissipate slowly.

Failures of very high dump faces can have significant consequences. At several mountaintop coal mines in the Rocky Mountains, large dump failures have occurred, through a combination of steeply sloping foundations and poor drainage. These failures have resulted in long toe runout distances, sometimes up to several kilometres. Figure 11 illustrates the geometry of an actual dump failure. Such occurrences are very dramatic and have serious safety implications, both from hazard in the toe areas and for personnel and equipment working on the crest. However, for economic reasons, such operations are still in current use and provided they are carefully monitored so that potential failures can be predicted, they can be operated safely.

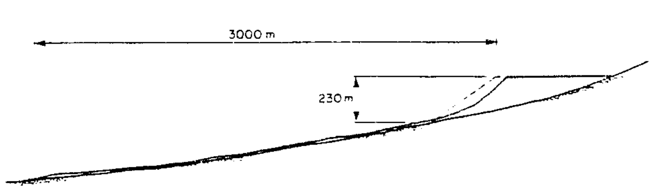

Figure 11: Geometry of Failure of a High Coal Mine Waste Dump in Canadian Cordillera

Overburden dumps formed of cohesive soils, or dumps built over cohesive soils, can be very troublesome as failure surfaces are more deeply seated. For this reason, adequate factors of safety must be maintained throughout dump operations, by limiting both the height of individual dump benches and the overall dump slope. A normal maximum bench height of 15 m is specified for overburden dumps.

In open-cast, coal mining applications, particularly on large, "Plains" dragline operations, waste is cast on the adjacent mined-out floor of the open pit. In this situation, spoil stability is required only for a temporary period while each "pass" of the mining operation is completed. However, stability is nonetheless critical as failures could cover unmined coal seams and result in production losses. In many typical sedimentary sequences, the beds immediately below coal seams may consist of very weak materials such as bentonitic clays. Planning of waste casting operations is a critical economic consideration in coal strip-mine design and requires careful geotechnical investigation.

Monitoring

Experience has shown that large slips of high waste dumps are always preceded by a period of slowly accelerating movements. By monitoring dump crest movements regularly, the safety of personnel and equipment can be safeguarded by temporarily suspending operations at a dump when increasing movements are noted.

The most common method of monitoring crest displacement is by the wireline method illustrated in Figure 12. Dump crest movements are monitored by periodically recording the movement of a weight suspended over a pulley on a tripod stand, placed at a stable location some distance back from the crest. Plots of rates of displacement vs. time are maintained and observations used to determine dump safety.

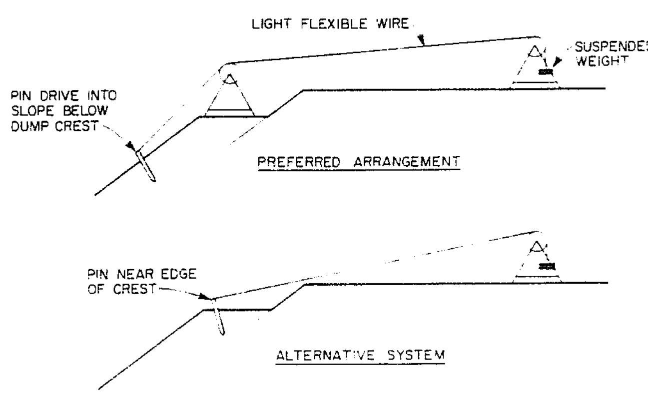

Figure 12: Typical Dump Movement Monitors

Other less commonly used dump movement monitoring methods include use of electronic distance measuring (EDM) laser equipment.

Environmental Aspects

Acid Mine Drainage - A major consideration in mine waste dump planning is the potential for oxidation of sulphides contained in waste rock. Such oxidation will lead to production of acid drainage and also, in some instances of coal mine refuse, to spontaneous combustion within the spoil pile.

The potential for acid mine drainage of waste rock can be determined by quantitative analysis of the sulphur content of the waste, which is then compared to the content of neutralizing components in the waste. Care must be taken in selecting representative rock types for testing. If the potential is determined for oxidation of only certain rocks within the overall mine waste, it is often possible to control placement in dumps so that acid-consuming rock types can neutralize acid producers. The basic requirements for sulphides to oxidize are to have an adequate supply of both air and water, so that design for prevention of acid production entails exclusion of air or water. Methods of treatment of potential acid producers include the following:

a) Runoff diversion
b) Capping dumps with impervious soils
c) Submergence under water
d) Avoidance of groundwater discharge areas
e) Collection and lime treatment of acid water

The latter alternative is to be avoided if at all possible, as treatment may be required long after mine operation ceases and will be an ongoing liability for the mining company or for the public.

Sediment Control - Waste dumps often generate significant sediment volumes which may enter downstream water courses. Sedimentation ponds are often constructed in streams downstream of waste dumps to allow settling of suspended solids.

Reclamation - Waste dumps usually are required to be reclaimed to support a stable vegetative cover at end of mine operations. Waste dump reclamation may often be carried out on final dump surfaces during the active mine operating life.

Regulatory agencies normally require that dumps be resloped, usually to 2 horizontal:1 vertical, or flatter, to provide a stable surface for vegetation. As discussed previously, the ease of resloping depends on the dump configuration chosen. Fine-grained, waste materials may be suitable for direct application of seed and fertilizer, while stockpiled overburden may need to be spread on coarser waste dumps to provide a suitable medium for vegetation. Coal mine wastes will usually weather readily to form a fine-grained veneer on the surface.

HEAP LEACHING

Introduction

Leaching of ores with cyanide has been practised for about 100 years, but the large-scale heap leaching currently being used in North American applications is a relatively recent development. Heap leaching with cyanide (usually sodium cyanide) is utilized for low grade ores with finely disseminated gold and silver mineralization. The process entails construction of a lined leach pad, heaping of the ore on the pad and sprinkling of cyanide solution over the ore. The pregnant solution is recovered from the base of the heap and directed to a recovery plant using either carbon or powdered zinc onto which the metals are adsorbed. Figure 13 presents a schematic illustration of the process.

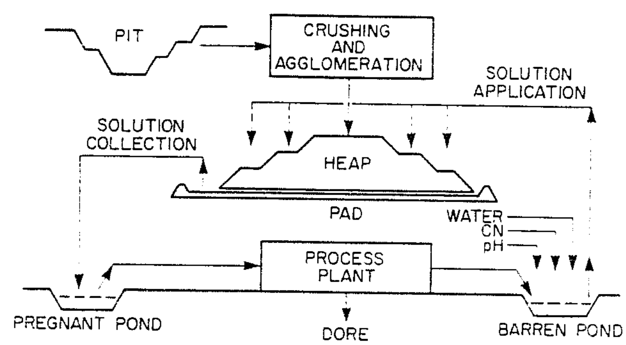

Figure 13: Schematic of Heap Leach Process

The process in principle is quite simple and for operations where large tonnages of ore can be handled at low unit cost, the method can be used to economically recover gold and silver values from very low grade ore.

Geotechnical engineers are involved in all phases of heap leach construction, starting with site investigations and carrying on through design to operational monitoring and ultimately to reclamation after cessation of operations. The many issues to be addressed include containment of operating solutions, stability of the ore heap, liner integrity, handling of precipitation and runoff, detoxification of the spent ore, and long-term reclamation procedures.

Heap Concepts

There are two types of heap leaching operations. The first utilizes a re-usable pad where a pad is loaded with ore, the ore leached for a few days or weeks, then the ore is detoxified and removed to a waste dump. In the second type the ore is left in place permanently and more ore is added over months or years. The re-useable pads are generally flat with just enough grade to get drainage and are used for ores where recovery is rapid.

Permanent heaps are used where recovery takes longer or where the recoverable values are low and rehandling cannot be justified.

Permanent heaps were initially constructed in the same manner as the re-usable ones - with pads sloping at about 1%, ditches along the edges to collect the pregnant solution, and adjacent lined ponds to store barren and pregnant solutions. In relatively flat country this type of heap, as shown in Figure 14, works well, but in hilly terrain it requires very large amounts of cut and fill to grade the pad area. In the last few years the valley-fill, heap concept has been developed. This method utilizes a cross-valley dyke and lining of the bowl formed by the dyke, as illustrated in Figure 15. The dyke stabilizes the heap and allows the storage of solution. In steep terrain, construction of large ponds is difficult and storage of solution in the voids of the ore can be utilized for ores that are structurally strong and relatively clean, and therefore do not require agglomeration.

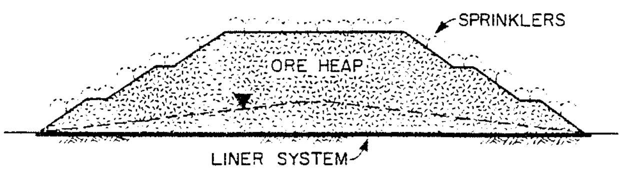

Figure 14: Flat Topography Heap Leach

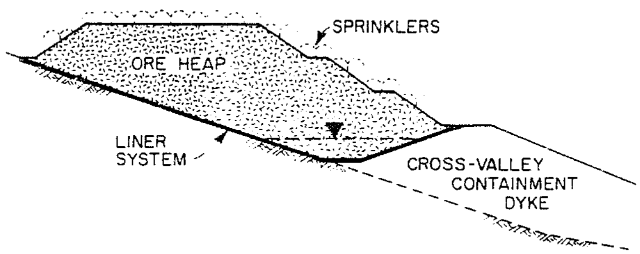

Figure 15: Valley Fill Heap Leach

Re-usable heap leach pads are generally limited in height to between 3 and 10 m and with an ore mass on the pad of under 20 000 tonnes. Permanent heap leach pads are much larger and currently pads are in operation that will ultimately contain ore in heights up to 120 m and in mass up to 50 million tonnes.

Recovery from ores generally increases as the ore size is reduced. The ore size required for economic recovery, which is determined from column tests and pilot heaps, can range in maximum size from 200 mm down to 15 mm.

Height of Heap

Most mining companies want to know the maximum height to which ore can be heaped in order to maximize the ore tonnage to pad area, ratio. For structurally sound ore, internal crushing begins to occur at heights of about 60 m. Laboratory tests on ores to measure their strength indicate that the strength begins to decrease when confining pressures reach the equivalent of a 60 m high pile of ore. This requires the use of flatter heap slopes for higher heaps. The permeability also decreases somewhat as internal crushing occurs and fines are generated.

For softer, clayey-ores the workable height is much less. Clayey ores generally need to be agglomerated to provide sufficient permeability for leaching. This can be done by pre-wetting the ore with cyanide solution, adding lime or cement and mixing by dropping onto successive conveyors resulting in an agglomerate structure of low strength. The structure of the particles agglomerated with water and cement is very weak and will break down when the heap becomes higher than approximately 5 to 7 m. After leaching, the ore becomes wet and muddy and loses strength. This type of ore should be placed in 5 to 7 m lifts, with each lift fully leached before more ore is added. Placement of a drainage blanket of harder ore between each agglomerated lift is required to maintain solution circulation. The slope of the ore heap must be sufficiently flat to ensure stability of the low strength ore. Slopes of 2.5:1 are common.

Some ores comprise structurally sound rock containing considerable fines which impede flow of solution. These ores can be agglomerated to provide a more permeable mass with fewer restrictions on lift height.

Water Balance

A water balance must be determined for each heap leaching operation. The water balance must account for all water entering, leaving, or being stored in the heap. The mining operator must know the water balance in order to determine the amount of fresh make-up water that may be required in dry periods and conversely the amount of storage that must be available in storms or winter periods.

In the western United States, many operations are in semi-arid areas, where water rights are fully committed and either have to be purchased at high cost or groundwater supplies have to be located. Evaporative losses can be very high in such areas, as much as 15 percent of the sprinkling rate.

In other regions, such as high mountainous areas, providing sufficient storage to handle spring runoffs and heavy rainfall may become the major concern.

To calculate the balance of inflows and outflows the precipitation and evaporation data for the area are determined and design parameters compiled. Column tests on the ore can provide data on the amount of solution in transit through a ton of ore under leach, and the moisture content after draining. A month by month spreadsheet of inflow and outflows is prepared with the key points being:

Inflow:
- Precipitation on catchment area

Consumption:
- Solution required to wet the ore to get flow established (3 to 10% range).

- Solution in circulation through the ore (up to 2%). In big heaps this can be a very large amount and of course stays in transit in the ore until a power or pump failure occurs or the operation is shut down for winter. It will take several days for the heap to drain.

Losses:
- Evaporation - from the sprayed area.

- Sublimation from snow cover on the heap in winter in colder areas.

For design purposes, both wet and dry periods, based on the available climatological data, are determined and the volumes of storage and makeup required to meet these design criteria calculated.

In the dry semi-desert areas under normal conditions, make-up water is required in every month of the year, so storage requirements are minimal. In colder, wetter climates where sprinkling is shut down in winter, accumulations of solution can be large with peak storage requirements coming just before spring startup. Some operators have considered removable covers over the heaps to limit the precipitation input. At this time covering of permanent heaps is not done but one tropical load/unload operation utilizes roofs on wheels rolled over the heaps to keep rain from diluting the solution.

It is important to provide surface water diversions to control runoff from getting on the pad so that the storage system does not get overtaxed. For conventional heaps this can be easily achieved with small ditches. However, for valley fill heaps with significant upstream catchments, major stream diversions are usually required.

In areas of moderate to high precipitation the inflows can exceed the outflows and the release of excess solution may be required. Detoxification of the solution before it can be released is necessary. One of several available methods of chemical treatment is used to accomplish this objective.

Solution Storage

Solution storage may be provided in the traditional way, using ponds constructed with synthetic liners, one pond is used for pregnant solution and the other for barren solution. Overflow ponds may also be provided to store excess solution during wet periods or emergencies. In mountainous areas or where topography is resticted, solution storage can be provided in the ore heap itself. For valley fill heaps this is straightforward as the ore containment dyke can be lined to create a solution reservoir.

The key parameters needed to calculate solution storage potential in the ore are porosity and residual moisture content. The porosity can be calculated from the density and specific gravity of the ore, and the residual moisture content can be measured in column tests. When the volume of residual moisture is subtracted from the gross porosity this gives the usable porosity, which is generally in the range of 12% to 30%, with 20% being a common value.

Liners

Liners are required to recover the gold dissolved from the ore to contain the solution chemicals, and to prevent contamination of the groundwater.

Early heap liners were often no more than the compacted ground surface. These were followed by compacted clays which generally worked well with careful quality control on selection, placement, and compaction to avoid stony patches or poorly compacted areas of high permeability. Two special problems that may arise with the use of clays are:

- some clay shales contain carbon that may absorb gold

- some clays may not be compatible with the leach solution and subsequently have their properties altered, becoming more permeable.

Geomembranes or synthetic liners are now commonly used, sometimes alone and at other times in conjunction with clay liners. There is a large array of choice of HDPE, LLDPE, PVC, CPE, CPER, Hypalon, etc., and each type has its advantages and disadvantages. The most commonly used liners are HDPE (high density polyethylene) and PVC (Polyvinyl Chloride). HDPE has a high puncture resistance, is resistant to all but a few organic compounds, is sunlight proof and is available in thicknesses up to 2.5 mm. It requires experienced crews to field weld the sheets together using extrudate or heated wedge techniques. PVC is more flexible and can be used where the base is expected to settle. It is not resistant to ultra violet light so cannot be left exposed permanently. PVC is less puncture-resistant than HDPE but is also less expensive and can be protected from puncture by using a geotextile or a layer of sand or fine ore. PVC is solvent seamed so can be installed by less skilled labour. PVC will crack with handling in temperatures not much below freezing and therefore cannot be used for winter installation.

Several types of liner systems are illustrated on Figure 16 and are described following:

1. Clay only
   Clay needs to be of low permeability. Minimum 300 mm thick placed in two

lifts to ensure coverage of any flaws. With a single liner there is no backup should a leak develop.

2. Geomembrane/Clay
With a double liner a backup is provided, and also the clay acts as a cushioning layer for the membrane.

3. Geomembrane/Drain/Clay
With this arrangement any leak through the membrane can be drained and collected to make the clay a "zero head liner" to protect the environment. Under low ore heights a geotextile can be used as a drain but under ore heights in excess of 30 m, sand should be used.

4. Geomembrane/Clay/Drain/Clay
This system is similar to the US EPA requirements for hazardous waste storage. The geomembrane/clay double layer provides good puncture and permeability characteristics and the drain ensures minimal or zero head on the bottom clay liner which protects the groundwater.

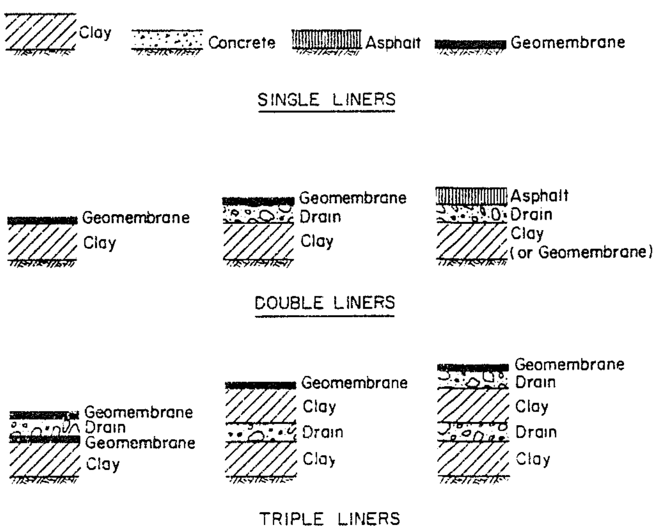

Figure 16: Types of Liner Systems

Generally, if solution is not going to be stored above the liner, a single liner may be suitable. However, many mining companies prefer to have a double liner for security, and many regulatory agencies consider a double liner to be necessary. For valley fill heaps, where solution will be stored above the liner and there is potential for steady leakage, a double liner system is required. A drain between the liners is desirable where the storage head above the liner is significant. The triple liner and drain is used where the significance of a leak is major or where the regulatory agencies dictate that any leak from the system constitutes failure and is cause for shutting down the operation.

Inspection and testing of liners being installed is important to ensure that a quality product is constructed.

Liner Design

For clay liners the permeability and strength characteristics can be determined using standard laboratory soil mechanics tests. Generally low to medium plastic clay is used as highly plastic clays have lower strength and can cause stability problems. Clay permeability should be 10^{-6} cm/sec or less and some regulatory requirements set 10^{-7} or 10^{-8} cm/sec as the required permeability. Additives can be used to lower the permeability of clays.

Synthetic liners must be resistant to the solution to be used, flexible enough to accommodate expected movements, puncture resistant, and ultra violet light resistant in areas that will remain exposed. The heap leach solution is a weak sodium cyanide having a very high pH. Such a solution is not harmful to the common liner materials.

Laboratory puncture tests are performed to check the performance of a liner system. A sample of the liner is placed in a cell, covered with a sample of the ore to be used and loaded to a stress equivalent to that which will occur in the design heap. This can be up to 2000 kPa. After 24 hours a check for punctures or near punctures is made and the requirements for protection of the liner are evaluated.

The larger sizes and sharper angled pieces of ore are the most likely to cause puncture. This is important to note as segregation of ore in placement can lead to greater likelihood of puncture. It is important that the first lift be limited to about 600 mm to avoid separation. Also the more uniform the gradation of an ore the more likely it is to cause puncturing.

Various geotextiles, generally non-woven, needle-punched polyester or polypropylene, are available to protect the liners. The staple fibre variety, that is short chopped fibres, are best for protection and come in various weights up to 550 gm/m^2. These simply keep the pieces of ore from penetrating the liner. Alternatively, a layer of 15 mm minus sand, or old tailings, or fine ore can be used. These alternatives can be evaluated in the load testing cell.

After selecting a workable liner system the friction coefficient between adjacent materials is assessed. This is very important to the overall stability of a heap, and the stability of ramps and initial ore placement for the valley fill heaps. Generally the lowest friction angles are found with the stiffest and smoothest membranes. Softer, more flexible membranes produce higher friction angles. Table I shows typical values of friction angle for commonly used materials. For flatland heaps the friction angle is not generally critical but slides have occurred due to low friction angles between adjacent materials. For PVC's the manufacturers generally produce one smooth side and one rough side. For PVC placed against clay the percentage difference in friction between the smooth and rough sides of the PVC can be high. Care must be taken to ensure that what is tested is in fact specified and supplied to avoid supplying materials that have frictional contact strengths that may reduce stability.

TABLE I

Typical Contact Friction Angles

Materials	Friction Angle
PVC rough/Clay	9.6*-26.2
smooth Clay	6.1*-25
PVC rough/Sand	25-27
smooth/Sand	21-25
PVC/Ore	33
PVC rough/Geotextile	23
smooth/Geotextile	21
HDPE/Clay	13
HDPE/Sand	17-26
HDPE/Ore	26-29
HDPE/Geotextile	7.3*-11.3

*Residual Value

The geomembrane industry is currently developing a range of membranes with rough surfaces specifically designed to assist stability.

Heap Stability

The stability of a heap is determined by the strength of the foundations, liners, ore, and the geometric shape of the heap. Where liner combinations are used, especially synthetic/clay or geomembrane/geotextile combinations, a low shear strength surface may exist at the contact between liners. This weak plane can control the stability of the heap and can result in the requirement that the ore slopes must be laid back at 3:1 to be stable. This problem is particularly acute in areas of high seismicity. In the past, owing to low friction angles between materials, some heaps built at an angle of repose have suffered edge failures under static conditions. Provided the friction angles of all the materials involved are determined, ore slopes that are stable under both static and earthquake conditions can be satisfactorily designed and constructed.

With a valley fill heap the containment dyke provides a stabilizing influence for the heap and because the valley fill dyke forms a reservoir to store the cyanide solution it has to be treated like a dam. Dykes are constructed with waste rock placed in lifts. Clean rock can be compacted either with the truck traffic or using a heavy vibratory roller. With clayey rock the water content has to be controlled and compaction is with either vibratory or tamping foot rollers. A filter layer of fine rock is placed on the upstream slope of the dam and the liner system is constructed on this filter layer.

Downstream slopes are determined by foundation strength and rockfill strength and are generally in the range of 2:1 to 3:1. Upstream slopes are set to a maximum slope of 3:1 for ease of construction. This is the steepest that liner slopes can be constructed without requiring that the men working on the liner be held by ropes. For clay liner construction, a 3:1 slope is approximately the maximum slope on which compactors can easily work.

For those heaps where a double liner consisting of a geomembrane over clay is used, very low friction angles may result. The low frictional resistance between the clay and the geomembrane can be greatly increased by placing a layer of sand between the two lining materials. This allows placing the ore at a steeper slope.

Solution Recovery

For flatland heaps the pregnant solution drains to a ditch along the low edge of the pad and flows to a storage pond. In the case of a valley fill heap there are two ways of recovering the solution. The first is to put a pipe through the dyke and liner and drain by gravity. This method is simple to construct but requires manual control in emergency or very wet conditions. Moreover, it requires a connection through the liner which can lead to leakage under higher heads. The alternative is to construct vertical wells, at the lowest point of the liner and install submersible pumps in the wells. Where the ore is clean and the pregnant solution is mostly clear, small diameter turbine pumps can be used. Where the leachate contains suspended solids slurry pumps, which require larger diameter wells, should be used. The vertical wells have the advantage of not compromising the liner and should the pumps fail no containment integrity is lost.

A collection system of perforated pipe is needed to collect the solution from the base of the ore and direct it to the well. Corrugated HDPE pipe can be used for this purpose and is available in diameters up to 600 mm. For ores where the permeability is not high, a drainage layer of higher permeability ore or a network of collector pipes is needed across the pad.

Reclamation

For reusable pads the spent ore is generally rinsed with water following cessation of leaching. The ore is next detoxified by circulating an oxidant followed by rinsing with water until the return water is of acceptable quality. Detoxified ore is then removed from the pad using loaders and trucks and hauled to a waste dump.

In permanent pad heaps the ore is not moved after placement in the heap. At the cessation of operations, the ore can be detoxified by a combination of circulating the solution, allowing natural breakdown of the cyanide in the sunlight, and the addition of oxidants. In dry climates a large proportion of the solution can be reduced by evaporation. When the solution is of acceptable quality it can be released. Liners for valley fill heaps where solution was stored in the heap, can be perforated by drilling from the surface of the heap to avoid long-term accumulation of water.

Ore slopes must be left in stable condition. Where the ore will degrade significantly with time, the reclamation designs must allow for this eventuality. Generally, overall slopes between 2:1 and 3:1 are used for reclamation design. Safety benches are used to break the slopes to avoid sheet drainage that could cause significant erosion.

Topsoil where available can be used to cover or blend with the ore surface to provide a rooting medium for reclamation.

Provision for permanent routing of runoff water must be made to direct flow around the heap.

CONCLUSIONS

The three broad areas of waste management that are covered by this paper, clearly illustrate the extensive scope of the geotechnical engineering services that are utilized by waste management activities in the mining industry. This work provides interesting, as well as challenging problems for the geotechnical engineer. Many of the projects are very large and waste management involves the design and construction of massive works whose safety is critically important to the health and welfare of people and property located downstream of the structures.

Handling of waste products, in many instances, is a major contributing factor to the total costs of the mining operations. Moreover, handling of waste products has become an environmentally sensitive issue which must be treated in a manner that satisfies both safety and pollution requirements. The major challenge to the geotechical engineer is therefore to develop solutions to waste management problems that are both economical and acceptable to the regulatory agencies concerned with safety and the environment. In the writers' opinion, this challenge is being successfully met as geotechnical engineers become more and more involved in all aspects of mining waste management.

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Geotechnical Practice in Petroleum Resources Development

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SUMMARY

This paper presents a review of the application of geotechnical engineering to the solution of problems related to the development of petroleum resources in Canada. The petroleum industry had its start in Canada 130 years ago but it was not until major pipeline construction projects initiated in the 1950's that what we now know as geotechnical engineering started to be applied to petroleum industry problems. The construction of pipelines, required specialized geotechnical expertise similar to other linear transportation facilities and the associated pumping stations or compressor stations encountered a need for specialized knowledge of soil dynamics. When planning for pipelines in the arctic was started in the late 1960's, geotechnical engineers had to address new problems related to permafrost, such as thaw settlement and frost heave, that increased geotechnical engineering input an order of magnitude beyond that required for conventional pipelines.

Among the greatest challenges presented to geotechnical engineers by the petroleum industry are the various developments of the Alberta Oil Sands. Geotechnical studies of oil sands were initiated in the late 1950's and early 1960's and have grown at an exceptional rate. Development of the oil sands has had a greater impact on geotechnical engineering than any other single undertaking in the history of Canada.

Canada has yet to have sustained production from an offshore reservoir but exceptional reserves have been revealed in the Arctic and off the East Coast. A substantial amount of geotechnical effort has been directed to engineering studies related to exploration and future development. A remarkable capability for marine geotechnical engineering in Arctic waters has been developed in Canada, primarily over the past decade.

Geotechnical engineers have a major role to play in almost every phase of resource development. World prices of oil today have rendered many of our frontier reservoirs uneconomical or only marginally economical. The challenge that faces the geotechnical profession in the coming years, particularly the R&D sector is to creatively develop means by which exploration and development of our declining resources can be done more economically and with greater efficiency.

RESUME

Cet article constitue une revue des applications du génie géotechnique à la solution de problèmes reliés au développement des ressources pétrolières au Canada. Les débuts de l'industrie pétrolière au Canada datent de 130 ans mais c'est seulement depuis que des projets importants de construction d'oléoducs ont été lancés dans les années cinquante,que ce que l'on appelle le génie géotechnique a commencé à être appliqué à l'industrie pétrolière. La construction d'oléoducs a fait appel à une expertise géotechnique particulière, voisine de celles qui s'appliquait aux moyens de transport par lignes. De même, les stations de pompage ou de mise en pression associées ont demandé une connaissance spéciale en dynamique des sols. Lorsque les études pour les olécoducs dans l'Arctique ont commencé, à la fin des années soixante, les ingénieurs géotechniciens ont été confrontés à des problèmes nouveaux reliés au pergélisol, comme le tassement consécutif au dégel ou le soulévement dû au gel, ce qui a amené la contribution du génie géotechnique un degré au-dessus des besoins traditionnels dans ce domaine.

Parmi les plus grands défis que l'industrie pétrolière a présentés aux ingénieurs géotechniciens on retrouve les différentes phases de développement des sables bitumineux en Alberta. Les études géotechniques des sables bitumineux en Alberta ont commencé à la fin des années cinquante, début des années soixante et se sont évolué à une vitesse exeptionnelle. Le développement des sables bitumineux a eu un impact plus grand sur le génie géotechnique que n'importe quelle entreprise de toute l'histoire du Canada.

Le Canada doit cependant s'appuyer sur une production soutenue à partir de réserves en mer mais un potentiel exeptionnel a été mis en évidence dans l'Arctique et au large de la côte Est. Un montant substantiel d'effort en géotechnique a été dirigé vers les études reliées à l'exploration et au développement futur. Une capacité remarquable dans le domaine de la géotechnique marine en eaux arctiques s'est mise en place au Canada, principalement lors de la dernière decade.

Les ingénieurs géotechniciens ont un rôle majeur à jouer dans pratiquement toutes les phases du développement des ressources. Les prix sur le marché international du pétrole ont rendu beaucoup de nos reservoirs potentiels non viables ou marginalement viables économiquement. Le défi que la communauté géotechnique a à relever dans les années qui viennent, particulièrement en recherche et développement, est d'étudier de manière créative les moyens de rendre l'exploration et la production de nos ressources sur le declin plus économiques et plus efficaces.

1. INTRODUCTION

The petroleum industry in Canada is approximately 130 years old, having started with the drilling of the first successful Canadian oil well in Oilsprings, Ontario in 1858. The Turner Valley Field in Southern Alberta (initiated in the early 1900's) was the first major oil field development but it was the Leduc discoveries in the late 1940's that triggered the construction of major pipelines in Canada and the application of, what is now known as, geotechnical engineering to the development of Canada's petroleum resources. Construction of pipelines that were needed to get the oil and gas to the consumers had to overcome many of the geotechnical constraints that faced other linear transportation systems. River crossings required the same considerations as bridges, including stability of riverbanks, protection against erosion and a stable foundation. Overland construction had to avoid swampy areas, unstable slopes and regions of high water table. Mountainous regions presented problems of steep terrain, snow and rock avalanches and even sporadic permafrost at high altitudes. Where it was uneconomical to go around these features, geotechnical engineering was applied to solve the problem. When pipelines from the Arctic were considered as a means of transporting oil and gas in the late 1960's, geotechnical engineers had to address new problems related to permafrost such as thaw settlement and frost heave that increased geotechnical engineering input an order of magnitude beyond that required for conventional pipelines.
At the same time geotechnical applications in the pipeline industry were increasing at an exponential rate, development of non-conventional oil reserves and frontier resources were to present Canadian geotechnical engineers with the greatest challenges in the history of their profession. Development of the Alberta oil sands has had a greater impact on geotechnical engineering than any other single undertaking in the history of Canada. Coupled with offshore resources exploration and potential production, the development of non-conventional petroleum resources has dramatically changed the practice of geotechnical engineering in Canada. Even in the depressed industry situation of 1986 and 1987, most geotechnical consulting firms in Canada have some involvement, either directly or indirectly, in solving geotechnical problems associated with getting more petroleum resources out of the ground or the problems of getting it to market once it is out.

The leading-edge geotechnology related to present day petroleum resource development lies with the oil companies and service industries. The Canadian geotechnical consulting capability for resource development, particularly the frontier resources, is probably as strong if not stronger than any country in the world. But the research and education sectors within government and universities have not kept pace. This has obvious serious long term implications, particularly for universities.

Very few people in the petroleum industry expect to see 40 U.S. dollars per barrel of oil within the next couple of decades. At present, much of our non-conventional oil is not economic. Even some well-established operations, initiated at a time when continuously rising oil prices were predicted, are still producing because it is probably more costly to shut them down, in the short term, than to keep them operating. Our most promising offshore resources at present require returns in excess of about twenty U.S. dollars a barrel to be economic.

There is no doubt that non-conventional reserves such as oil sands and heavy oil and arctic and east coast offshore resources will be needed in the coming decades. There is also no doubt that if Canada does not place a high priority on the development of these reserves, we will at some time again face the same outrageous costs that we faced in the 1970's and early 1980's for imports from those countries with immense low cost developable resources, such as the middle-East countries.

The challenge to geotechnical engineers at this time is to develop techniques or to contribute to the technology needed to get oil out of the ground and to the market more economically than can be done with present methods. Investment in research today will lead to more economical energy for the consumer tomorrow. More efficient methods of extracting energy from the oil sands, more economical methods of producing from offshore reservoirs, better recovery from conventional sources, economical extraction of heavy oils from the ground are all engineering challenges for innovative geotechnology. Technology transfer from industry to university is essential for proper training of engineers and scientists that will be needed to meet the research and development challenges of the coming decades. Research infrastructure in the universities must be updated, broadened and refocused to meet these challenges. A new or revitalized partnership between industry and universities is essential.

This paper provides a brief review of the historical development of geotechnique within the petroleum industry, the technical achievements and the present challenges.

2. EARLY GEOTECHNICAL INVOLVEMENT IN PETROLEUM RESOURCE DEVELOPMENT

It is doubtful that geotechnical considerations had much influence on the first large petroleum resource developments in Canada. After all, not only had the word "geotechnical" not been invented but soil mechanics and foundation engineering had yet to crystalize as a discipline with its own technology. Nevertheless, the engineers and surveyors involved with the development of resources like those of Turner Valley, and later Leduc, showed an understanding of terrain and a common sense, similar to that shown by the early railroad and highway engineers and builders. It is much less tempting to move mountains when horse power is limited to horses or small trailers, than it is when huge pieces of earth moving equipment can be readily mobilized anywhere in the country. People worked with the terrain, avoiding large cuts or fills and unstable soils. So it was not until the 1950's that soil mechanics and foundation engineering was first applied to solving problems related to petroleum resource development. As within many of the engineering disciplines, one person played a dominant role that was to set a standard of practice for decades to come. That person was Dr. R.M. (Bob) Hardy who was to remain at the leading edge of geotechnology related to petroleum resource development until his death in 1985.

Although problems with slope stability, dykes or access roads occasionally demanded the skills of the soils engineer, it was the problems with structures and foundations for compressor stations and pumping stations associated with pipeline systems that required new skills in soil mechanics. Those new skills were related to the determination and understanding of the dynamic properties of soils and the analysis of potential vibrations. Some of the compressors, which became known as "one-lungers" were very unstable as they operated at a frequency that often coincided with the soil-foundation resonant frequency. The resulting vibrations were very damaging to connecting pipes and to the structure itself. When the operating unit was housed in a steel frame building with metal cladding, as was very common, the resonant foundation could be heard for miles away as the building acted like a huge bongo drum.

It became necessary to develop design guidelines that could help avoid system resonance before research had been carried out to provide an engineering rationale. One early approach was to ensure that the foundation block was of a size to be at least some minimum multiple, usually 2 times the weight of the compressor unit. This approach virtually ignored soil conditions. Another approach was to use piles, often drilled cast-in-place concrete. Neither of these foundation systems always worked for reasons that are now reasonably well-known. Widespread use was also made of a design chart relating reduced natural frequency to the foundation area, assembled by Dr. Hardy, drawing on work presented by Tschebotarioff at the Symposium on Dynamic Testing of Soils in connection with the fifty-sixth Annual Meeting of the American Society of Testing and Materials in July, 1953, as well as data from the personal files of Dr. Hardy. The approach used was to adjust the area of the foundation, and hence the contact pressure until the points obtained for the range of operating frequencies fell well away from the line representing the critical resonance condition. If they fell to the left of the line, the unit would not go into a resonant condition. If they fell to the right, it indicated that the unit would pass through a resonant frequency but it would not vibrate excessively at the range of operating frequencies. The design method worked well for the one-lungers or those units with a predominantly horizontal or vertical vibration mode. But as compressors and pumps became more complex with greater unbalanced forces resulting in rocking and torsional vibration modes, the chart could not be used. By this time, more sophisticated analytical methods and soil testing techniques were emerging and leading to the very detailed computer aided analysis and design techniques for dynamically loaded foundations that we have available today.

3. GEOTECHNOLOGY IN PIPELINES

Although several pipelines had been built in western Canada and the Territories prior to 1950, it was not until the Trans Mountain Pipeline was built in the 1950's that what we now know as geotechnology started to be incorporated as an integral component of pipeline engineering and construction. During this period, Dr. Hardy recognized the need for another emerging discipline, that of river engineering. He therefore brought in a colleague from the University of Alberta, Dr. Tom Blench to assist with the design of pipeline river crossings. This led to the formation of T. Blench and Associates and subsequently Northwest Hydraulics in the early 1970's, one of the premier river engineering specialist companies in the world and one which would play a major role in most of North America's major pipeline projects.

During the nineteen fifties, sixties and seventies most of the geotechnical input to pipelines was provided by Bob Hardy and Tom Blench. Dr. Hardy, sometimes accompanied by Blench, would carry out an annual inspection of the West Coast Pipeline System and would advise on maintenance requirements, particularly for problem slopes and river crossings. He also consulted on route selection and advised on other geotechnical aspects for new pipelines.

Much of the early experience with pipelines and the innovative methods of dealing with geotechnical problems was presented by Murray Harris and Bob Hardy at the 6th Western Region Conference of the Engineering Institute of Canada in Edmonton, Alberta in February 1980. The paper, entitled 'Geotechnical Considerations in Pipeline Failures' sets out a rational method of analyzing geotechnical loadings on pipelines.

Planning for the first major arctic pipeline started in the sixties when marketable reserves were discovered at Prudhoe Bay in Alaska. At first it was proposed that a conventional buried pipeline be built across Alaska through the permafrost areas and all of the pipe was ordered and stockpiled. Had construction proceeded as first intended, it would have been a major economic and ecological disaster. A series of seminars put on by permafrost experts of the US Geological Survey drew attention to the problems associated with building pipelines in permafrost and led to a delay in construction until a regulatory body was in place to ensure that the design was properly carried out. It would require four years of intense engineering design and by far the most extensive series of site investigations ever carried out for any project before construction started. Several Canadian engineering firms and a number of Canadians working for American firms were involved with the project. Northwest Hydraulics of Edmonton was responsible for all the river engineering work.

The Alyeska Pipeline was a remarkable technical achievement. It was initially estimated to cost about 900 million dollars but its final cost was close to ten billion dollars. The importance of geotechnical engineering in arctic pipelines was indicated by the project manager, Mr. Moolin, who was reported by the 'Engineering News Record' to attribute most of the cost overrun to be due to "negative geotechnical surprises!"

It was during the construction of the Alyeska Pipeline that extensive engineering studies were being carried out for a number of Canadian arctic oil and gas pipelines. Canadian Arctic Gas Pipeline Limited (CAGPL) planned a large diameter pipeline from Alaska to the Mackenzie Delta then up the Mackenzie Valley and into Alberta. Foothills Pipelines Limited filed for a similar route in the Mackenzie Valley but then switched to a route parallel to the Alaska Highway with a lateral from the Mackenzie Delta, following the route of the Dempster Highway. Polar Gas studied alternative routes for gas pipelines along both the east and west sides of Hudson Bay and subsequently the Mackenzie Valley as well. The Beaufort-Delta project studied a potential oil pipeline in the Mackenzie Valley from the Delta to Alberta. An oil pipeline for a similar route had previously been studied by a group known as Mackenzie Valley Pipeline Research Limited (MVPRL). Even a railroad in the Mackenzie Valley was considered.

All of the potential pipeline projects triggered extensive geotechnical studies throughout the Arctic. The studies advanced knowledge of permafrost terrain and geotechnical properties of frozen soils, thawing soils and freezing soils by an order of magnitude in a very short period of time. The studies and research related to arctic pipelines also produced major advances in our understanding of problems not specific to the Arctic such as frost action and construction over organic terrain.

Geophysical exploration techniques for locating shallow permafrost using resistivity and electromagnetic methods (EM) emerged rapidly during this period. Innovative EM methods proved to be particularly effective in mapping frozen ground. These techniques are now used routinely in permafrost regions and also in unfrozen terrain for mapping gravel deposits, depth to bedrock and so on. Several geophysical methods of determining **in situ** properties also advanced significantly during this period, such as cross hole seismic and down hole shear wave generators. Much still remains to be done on correlation of geophysical measurements with the significant engineering properties of soils. None of the proposed pipelines were built, but in 1985, Interprovincial Pipelines Limited completed an oil pipeline from Norman Wells in the Northwest Territories to Zama Lake in Alberta. This was the first pipeline project to make use of the technology developed by the many previous studies carried out for arctic pipelines. This pipeline will likely be either looped or parallelled and extended to link arctic oil and gas from the Beaufort Sea and Mackenzie Delta to southern Canada.

Another challenge yet to be faced in developing frontier resources is the successful construction of offshore pipelines in the arctic offshore or east coast. Geotechnical considerations, particularly related to ice scour, will play a major role in the design, construction and operation of these installations.

4. OILSANDS

The development of Alberta oilsands has probably had the greatest impact on geotechnical engineering of any single undertaking in the history of Canada. Studies of engineering properties of the oilsands undertaken in the late nineteen fifties and early sixties indicated that engineers were dealing with an entirely new material with behavioral characteristics unlike any other material found in the earth's crust. The work at that time was directed to an open pit mining operation known as the Great Canadian Oil Sands (GCOS) Project, now known as Suncor. Once again it was Bob Hardy leading the way, providing most of the geotechnical input to the development of the mining concepts and the tailings disposal scheme. By any standard, the Tar Island tailings disposal scheme for Suncor, at that time the largest tailings disposal system in the world, was an outstanding engineering achievement.

It was not until the Syncrude Project came on stream that involvement of geotechnical engineers increased by almost an order of magnitude. This was necessitated when a decision was made to use a dragline mining technique rather than the bucket wheel operation as used by GCOS at the time. The largest draglines in the world were put to work in excavating the oil sand from the top of the formation in a single cut. This mining technique requires a carefully engineered working surface with considerable attention directed to geotechnical details but most importantly, it requires continuous monitoring of the slopes, 24 hours a day, every day of the year. The tailings disposal from the project required over 25 miles of dykes to contain the slimes produced as a by-product of the hot water process used to extract the oil from the sand. The amount of sand tailings produced each year is equivalent to the total amount of material used to build the Gardner Dam in Saskatchewan.

The engineering properties of oil sands and the products of the extraction process continue to present engineering challenges that occupy the full attention of a large number of geotechnical engineers. The oilsand, which Dr. Hardy referred to as a "four phase system", is without doubt one of the most interesting materials encountered in geotechnical engineering. It is an exceptionally difficult material to sample in a relatively undisturbed state. Its behavior is temperature dependent. It has free gas in the voids and gas in solution in the water and bitumen and thus the behavior is very dependent on the confining pressure. Detailed laboratory studies of the material properties continue and will become even more important in the future as other mining techniques are initiated. The geotechnical group at the University of Alberta are world leaders in this technology. The slimes problem in the tailings disposal ponds is one that is also receiving considerable attention from engineers throughout North America with respect to long term environmentally acceptable means to stabilize the material.

The amount of the oil sands amenable to open pit mining operations represent only about 3% of the total deposit. This technique can be used economically for a depth of approximately 200-300 feet below ground surface. Below a depth of about 1000 feet, in **situ** methods such as the "huff and puff" steam injection method can be used. A major challenge lies in economically developing the pay zones between depths of about 300 to 1000 feet and geotechnical engineering will have an important role to play. Mine assisted techniques will likely be used wherein shafts are sunk into the ground and **in situ** stimulation techniques such as steam injection will be applied from tunnels, allowing gravity drainage to bring the oil back to the tunnels. Underground mining of the oil sands is also a possibility.

In **situ** techniques for deeper deposits of heavy oil such as the Esso Cold Lake Project also make use of geotechnical engineering skills in understanding stress state, fracture patterns, thermal aspects and reservoir behavior. The development of cost effective techniques for the extraction of oil beyond surface mineable limits will present remarkable opportunities for ingenious geotechnical engineers in the future.

5. OFFSHORE RESOURCES

Exploration drilling over the past two decades has revealed exceptional reserves in the Canadian waters of the arctic and east coast. The recent lifting of a moratorium on drilling off the west coast will likely lead to the discovery of reservoirs in that offshore region as well. Huge reservoirs in the Hibernia Area off the east coast and in the Amauligak field in the Arctic are at present in the advanced planning stage for development. Their economic borderline is about 20 U.S. dollars a barrel. In spite of present world oil prices, given the state of Canada's conventional reservoir reserves and cost of non-conventional fields, it is an almost certainty that Canadian consumers will be using fossil fuels from the Arctic and eastern offshore regions before the end of this century.

The major difference that distinguishes most of the Canadian offshore petroleum reservoirs from those that have been developed elsewhere in the world is that most of the Canadian waters on the east coast and all of the arctic waters are covered by or invaded by ice for part or all of the year. The properties of ice, and the interaction of ice with structures, vessels and the seabed becomes a dominant geotechnical consideration in the design of production facilities for these regions.

Over the past decade, an extensive amount of geotechnical engineering work has been carried out in the Canadian arctic waters, particularly in the Beaufort Sea. This work has included site investigations incorporating very sophisticated **in situ** testing and sampling techniques as well as laboratory testing and experimental modelling.

Most of the exploration work in the Beaufort Sea has been carried out by drilling from artificial islands. These started as surface piercing gravel or sand fill islands usually made with material dredged from the seabed although the first islands were constructed from gravel hauled over the ice from shore. The artificial island concept was pioneered by Esso in the 1970's. As exploration extended into deeper waters, sand islands became impractical because of their very flat slopes and hence very large quantities of sand required. This necessitated a new breed of artificial islands which incorporated dredged fill with a structure. The first hybrid island to be built and to be used successfully as a drilling platform was the Tarsuit Island. The structure consisted of four large concrete caissons that were floated into position to form a large octagon. They were then ballasted and sunk to sit on a previously dredged berm. The interior of the octagon formed by the caissons was filled with sand dredged from the seabed. The hybrid island constructed by Dome Petroleum worked very well and it was soon to be followed by other models developed by Gulf, Esso and Dome. The Molikpaq, a mobile arctic caisson developed by Gulf, is a large steel hull with a central core that is filled with sand after the hull is ballasted and sunk to a prepared berm surface. The Esso caisson retained island (CRI) is somewhat similar in shape and concept to the Tarsuit Island except that the caissons are made of steel. It is designed to create a rubblemound of ice around it whereas the Molikpaq is not. The Dome single steel drilling caisson (SSDC) platform is an entirely new concept. It is a modified ship shape that is ballasted and sunk to a prepared berm.

These platforms have been deployed several times and in each case monitoring of soil behavior, ice loading and structural response has been carried out. The monitoring has yielded an exceptional body of geotechnical data. Indeed, every aspect of the system requires a substantial amount of geotechnical engineering input. The site investigation usually involves drilling of a number of boreholes, sampling and **in situ** testing. Site preparation and berm construction requires careful scrutiny and quality control testing by geotechnical engineers. Placing of the sand core requires the same attention to detail and finally, the instrumentation, monitoring, analysis and interpretation of the performance data are major undertakings.

Recently Esso has successfully completed the first well in the Canadian Beaufort Sea drilled from a platform constructed from spray ice. This is a material somewhat similar to the material produced by snow making machines on ski hills. The ice is made from seawater sprayed in the atmosphere where the droplets freeze and fall to the ice covered surface. As the spray ice mound grows, the ice surface sinks and eventually contacts the seabed. The surface is then levelled for the drilling rig. Similar structures were constructed in the American Beaufort Sea prior to the Esso island with most of the design, testing, quality control and construction supervision being provided by Canadian geotechnical consulting engineering firms. Spray ice is one of the most interesting new construction materials to emerge in the arctic offshore scene and it will no doubt be widely used in arctic regions in the future, providing both opportunities and challenges to the Canadian geotechnical community.

Much less geotechnical engineering work has been done off the east coast of Canada, mainly because of the very different exploration methods and a less complex geotechnical environment. Jack-up rigs now require test holes before they are positioned but most of the exploration drilling is done from semi-submersibles. Geotechnical site investigation for site selection work has been carried out by Mobil Oil Canada for the Hibernia gravity based structures and detailed final site investigation is planned. Foundation conditions within the Hibernia area are very good and no unusual problems are expected. As development of other fields occur in the future, more complex geotechnical problems will have to be dealt with, particularly, for areas of the continental slope and some of the potential reservoirs off the coast of Labrador.

6. THE CHALLENGES OF THE NEXT TWO DECADES

The greatest uncertainty in the oil industry today in 1987 is the price of oil. Most of our frontier and non-convention petroleum resources are not economical at 15 U.S. dollars per barrel. On the other hand our conventional reservoirs, the ones that can be produced economically for that price, are declining rapidly and our reliance on imported oil is increasing at a corresponding rate. It does not take an economic wizard to understand that it will not be too many years before we and our major trading partner to the south will be facing exorbitant prices similar to the price that had to be paid for oil and gas during the last period of short supply. Canada needs a secure source of petroleum resources and there are no sources more secure than those within our own borders. But we also must be able to afford our own resources. The challenge facing engineers now is to develop means of producing affordable petroleum supplies from our frontier regions and non-conventional reserves.

Geotechnical engineers have a major role to play in every phase of more cost-effective resource development. Site investigations for offshore projects are exceptionally expensive. Better use of geophysical methods, coupled with the currently available **in situ** testing techniques, could not only lead to more economical programs but also to better coverage. We are currently not constrained in our ability to measure engineering properties in the laboratory or to develop mathematical models to analyze designs. We are constrained by our limited understanding of geology, geological processes and geotechnical properties as well as the environmental loads, the interaction of structures with those loads and the response of foundations. When we lack information or confidence in the data we do have, designs become more and more conservative and less economical. Hence reservoirs that are economically marginal become uneconomic. This is the current situation in the Canadian Beaufort Sea. We cannot only rely on increased oil prices to bring these resources into a producible economic range. The challenge to the geotechnical community, particularly the R&D sector, is to creatively develop means to improve efficiency and to bring the economics into the producible range.

Geotechnical Practice in Northern Development

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SUMMARY

The practice of geotechnical engineering has played an important role in development of Canada's north. Particular challenges have included dealing with permafrost, ice as a structural medium and ice or snow loads on structures. This paper addresses engineering for permafrost, drawing upon landmark experience with community development, transportation infrastructure, and development of northern resources. The evolution of design principles for foundations and the importance of predicting thermal response of the soil to any proposed development is emphasized. The paper is illustrated by reference to case histories drawn from the authors experience.

RÉSUMÉ

La pratique du génie géotechnique a joué un rôle important dans le développement du nord canadien. Les défis particuliers étaient reliés au pergélisol, à la glace en tant que matériau structural, ainsi qu'aux charges de neige et de glace sur les structures. L'article traite du génie du pergélisol, des évènements marquants dans le développement des communautés et des infrastructures de transport, et dans l'exploitation des richesses naturelles. On insiste sur l'évolution des principes de conception des fondations, et sur l'importance de prévoir quelle sera la réponse thermique du terrain à un projet de développement. Les illustrations de l'article proviennent des dossiers de l'auteur pour diverses études de cas.

INTRODUCTION

Civil engineering has been described as the art of directing the great sources of power in nature for the use and convenience of man. In the north, the forces of nature are exacerbated by a harsh climate. In this day of ever increasing specialization we have evolved a new discipline of geotechnical practice termed "cold regions geotechnology". This phrase encompasses civil engineering where ice, snow and permafrost are dominant factors. The purpose of this paper is to describe the evolution of cold regions geotechnology, what is unique about it and how it is practiced today. More importantly, however, is how this evolving technology has benefited the quality of life in the north and has facilitated development of our natural resources.

Civil engineers commonly refer to "the north" as the part of Canada where permafrost occurs, which includes about 50% of our country. The southern limit of permafrost is generally coincident with a mean annual air temperature of -1°C. This includes both territories and developed regions in Alberta, Manitoba, and Quebec. The winters are long and dark, resulting in a very short construction season. Construction planning is also controlled by the great distances and limited transportation infrastructure. This paper will primarily deal with design and construction on permafrost terrain. Design and construction of winter roads and sea ice loading on structures are also important topics in cold regions geotechnology, particularly with respect to exploration and development of arctic offshore resources; however, space does not permit an exhaustive treatment of the subject.

THE IMPACT OF PERMAFROST

Permafrost, or perennially frozen ground, is a condition of the soil wherein it remains at a temperature below 0°C year around. Probably the most significant parameter characterizing permafrost is the quantity and distribution of ground ice. Soils with ground ice in excess of that required for saturation will display significant settlement, low strength and high compressibility if they are allowed to thaw. The presence of ground ice can be related to geologic history of the terrain. It occurs as pore ice, lenses and much larger accumulations referred to as massive ground ice.

The simple fact that permafrost is not a soil type such as sand, silt or clay but a soil condition, means that its properties and behavior do not lend themselves to generalization. For example experience gained working with permafrost soils of the Mackenzie Valley can be misleading if it is applied directly to solution of problems in Yukon or on Baffin Island. There has been a tendency to generalize and develop "rules of thumb" when dealing with permafrost. This has led to dangerous misconceptions such as the following two examples:

a. "Bedrock always provides a competent foundation". Permafrost bedrock outcrops that appear dry and intact on the surface often contain ice in fractures and fissures. Overlooking ground ice in rock has jeopardized water supply dams and heavy industrial buildings in the Canadian North.

b. "Permafrost sand and gravel is free of excess ice". Pore ice is seldom visible in sand and gravel but some deposits have been found to contain extensive ice wedges and very large bodies of massive ground ice.

Ground ice is present in virtually all frozen soils and rock; therefore, it is incumbent upon the geotechnical engineer to structure a site investigation program that will determine its nature and distribution at any particular site. Near continuous coring of the frozen ground is required to allow logging of the soil and ice type. This has led to development of innovative drilling and sampling equipment such as the light weight auger coring rig shown in Figure 1. This unit, described by Innes (1984), can be transported between sites with a light turbine helicopter (Bell 206) or by Twin Otter aircraft. Cores of ice-rich fine-grained soils (Figure 2) are readily obtained with an auger core barrel originally developed for coring ice. Where boulders or rock are encountered it may be necessary to use a diamond coring drill (Figure 3) with added capability to refrigerate the drilling fluid such as described by Roggensack (1977). A core of frozen bouldery till containing ground ice is shown in Figure 4.

Permafrost mechanical properties can be related to temperature and soil composition. This introduces a need to know the current ground thermal regime, what factors influence it at a particular site and how to predict any changes that may be caused by development. Ice and water co-exist in frozen soil with the ratio depending upon temperature and impurities such as salt in the porewater. The extent of unfrozen water in frozen soil influences such important characteristics as rate of thaw, frost heave potential and deformation under constant load (creep). Subsea permafrost and coastal permafrost below the limit of marine trans-

Figure 1 Heliportable auger drill developed for pipeline route evaluation in the Mackenzie Valley.

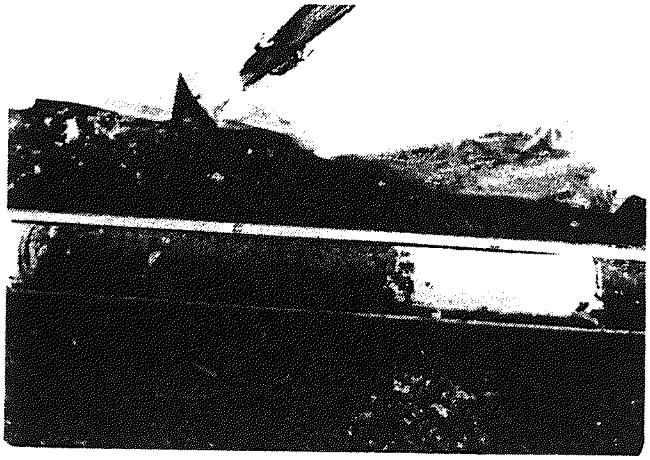

Figure 2 Sample of ice-rich, fine-grained soil obtained with a modified ice auger core barrel.

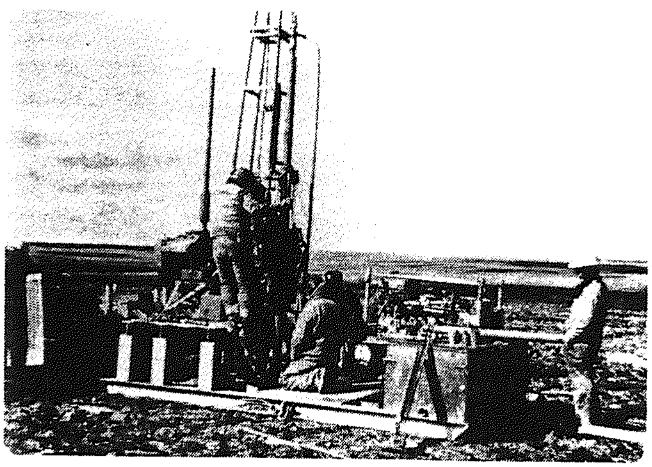

Figure 3 Heli-portable drill with mud refrigeration unit used for coring frozen bouldery till on the Polar Gas Pipeline route.

gression may contain saline pore ice with salt concentrations sometimes in excess of normal sea water. The detrimental aspects of saline permafrost in coastal communities has been identified recently because of its adverse impact on foundation design.

We tend to be somewhat cavalier in our approach to dealing with cold continuous permafrost where modest temperature changes are normally of little consequence. However, where soils are close to thawing, near the southern limit of permafrost or in the submarine environment, a clear understanding of the influence of even small temperature changes on behavioral characteristics is essential.

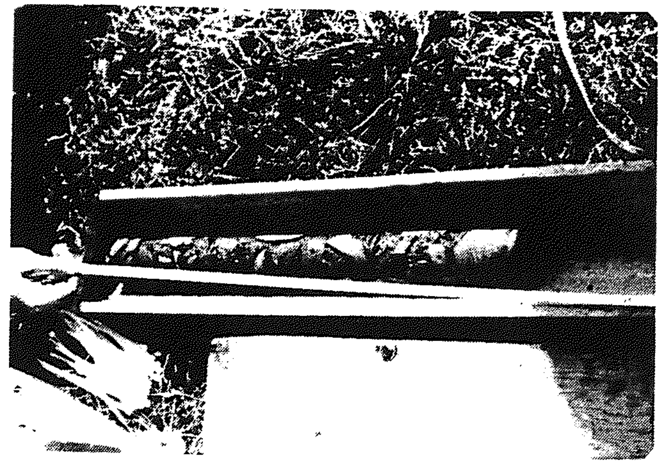

Figure 4 Diamond drill core of permafrost till from the Central District of Keewatin.

COMMUNITY DEVELOPMENT

In the early 60's I attended the first public showing of a film prepared by the National Research Council on the construction of Inuvik, N.W.T. It was at the National Museum in Ottawa and was followed by comments by Dr. R.F. Legget. Clearly, the construction of Inuvik was a milestone in development of cold regions geotechnology. The concept of elevating buildings to provide free air passage beneath the floor preventing thaw of the underlying permafrost was refined at this time. Also, this marked the first large-scale use of piles frozen into the permafrost, requiring development of innovative design and construction techniques.

Architectural and structural limitations imposed by elevated buildings have forced geotechnical engineers to consider alternatives for this simple but effective heat exchanger, particularly for industrial buildings with heavily loaded floors. Improvements to methods of predicting thermal response of the underlying permafrost allowed development of ventilated pad foundations and slab-on-grade foundations were the permafrost is sustained by inserting heat pipes. These foundations must be designed to compensate for heat loss from the building to the ground and must offset the effect of covering the ground surface with a warm structure. An early installation of this type beneath a school in Ross River, Yukon, is shown in Figure 5 (Hayley, 1981). This passive system of extracting heat from the ground in winter and dissipating it to a cold atmosphere is an offshoot of technology developed for equilibration of temperatures on orbiting space capsules. The first large-scale use of heat pipes was to improve the load carrying capacity of piles supporting the Trans Alaska Pipeline by lowering ground temperature at the pile-soil interface.

Irrespective of the type of heat exchanger chosen for construction on permafrost, adherance to the cardinal rule of northern projects is essential to success; "keep it simple" (KIS). Systems that require seasonal maintenance must be avoided. This important principal of northern practice can be aptly illustrated with the ventilated pad foundation shown in Figure 6. This common foundation system, typically used for supporting warm oil tanks required that caps be placed on the vent pipes each summer to eliminate warm air flow through the foundation. This has the overall effect of reducing summer thaw. On several occasions, I have observed that some caps have been left on in winter and some were not put on in summer which is directly contrary to the designers intent. Although it used to be common practice to specify such operating constraints, a prudent designer that adheres to the KIS principle would now use available geothermal analyses to design a free flow, minimum maintenance system. The design would be driven by service requirements, ground conditions, constructability and economics. The last three factors are location dependent, thus the northern geotechnical engineer must have a broad knowledge of the region in which he is working in order to choose an appropriate building foundation system.

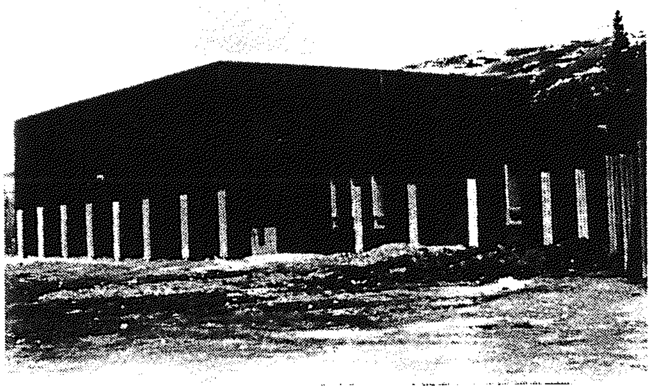

Figure 5 Early application of heat pipes as a heat exchanger beneath a slab-on-grade foundation over permafrost at Ross River, Yukon.

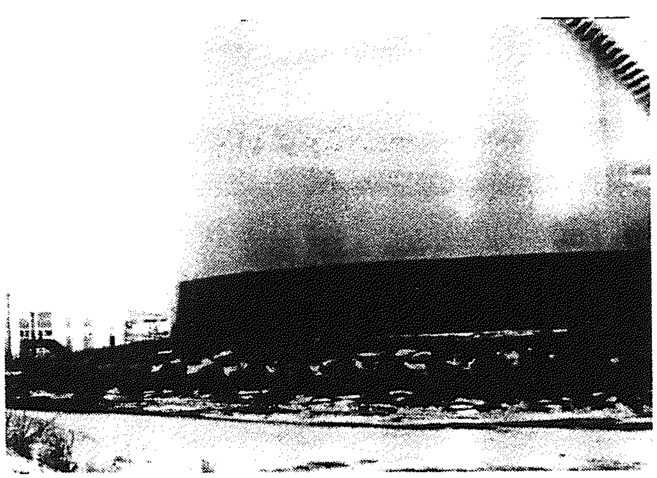

Figure 6 Ventilated pad foundation showing lack of attention to seasonal installation and removal of vent pipe caps.

The geotechnical engineer has played a major role in the evolution of utility systems for northern communities. Early systems such as the Inuvik utilidor avoided interaction with permafrost by use of an above ground structure. These linear structures that connect the buildings are generally considered undesirable obstructions by northern residents because they interfere with roads and pedestrian traffic. This has encouraged community planners and municipal engineers to adopt below ground systems even at greater initial capital cost. One system, described by Zirjacks and Hwang (1983) was installed in permafrost at Barrow, Alaska in 1982. A full utilidor containing water supply, sewerage and heating piping was placed in a large wooden box structure that was buried in cold ice-rich permafrost. This ambitious (and very costly) design was only possible because the geotechnical engineer was able to predict long-term ground temperatures in the vicinity of the buried structure. Judicious use of insulation allowed the designer to limit heat loss such that maximum seasonal ground temperature was within limits established at the outset to ensure satisfactory performance.

Flexible piping materials coupled with factory applied insulation have allowed direct burial of sewer and water piping in permafrost. A complete system of this type was installed in Dawson, Yukon in 1979. The polyethylene pipe system replaced wood stave piping that had been in service for more than 50 years. Dawson is situated on a floodplain on the Yukon River where permafrost organic silt is ice-rich and particularly unstable after thaw. The Dawson silt is also noted as a hazardous material because it heaves appreciably during winter freezeup. Particular attention had to be given to pipe stability, stresses caused by freezing or thawing soil, and performance of the backfill. This was a pioneering project from which much was learned about direct burial of piping systems in permafrost. With financial assistance from the Institute for Research in Construction, NRC, ground temperature data was collected adjacent to the operating system that was invaluable for refining the design of a system extension constructed in 1986 (EBA, 1983).

TRANSPORTATION INFRASTRUCTURE

The Hudson Bay Railway, completed in 1929, was the first major transportation facility built over permafrost in Canada. It links The Pas, Manitoba to Churchill on Hudson Bay and serves as an alternate route to port for prairie grain. The route traverses the Hudson Bay Lowlands, one of the largest continuous expanses of peatland in the world. Permafrost is discontinuous but widespread in the peat and underlying mineral soils with its frequency of occurrence increasing northward until it becomes continuous just south of Churchill.

Construction of the railway grade caused minor changes to the ground thermal regime. Compression of the peat, drifting snow, and interruption of drainage have caused permafrost thaw, particularly at the transitions from frozen to unfrozen terrain. Thaw caused progressive settlement of the granular subgrade at recurring sites termed sinkholes by CN maintenance personnel. A typical sinkhole, such as the one shown in Figure 7, may require lifting the track as much as 150 mm each year.

Ongoing research initiated in 1976 has been directed towards finding an economically viable solution to the maintenance problems that threaten the very future of the railway. Experimental test sites, such as the one shown in Figure 8 (Hayley et al., 1983) have demonstrated that individual sinkholes can be stabilized by insertion of heat pipes into the railway grade at the transition from nonpermafrost to permafrost terrain. The heat pipes will enhance heat removal in winter, altering the net annual heat balance sufficiently to stop progressive degradation of the permafrost. Precise location for the heat pipes may be determined using ground penetrating radar. Experimental testing has shown that a continuous profiling ground radar system, such as described by Annan and Davis (1976), is capable of locating the permafrost transition within an active sinkhole to an accuracy that will minimize the number of heat pipes required. This application of new technology to a maintenance problem that is currently out of hand may ultimately allow rehabilitation of the railway.

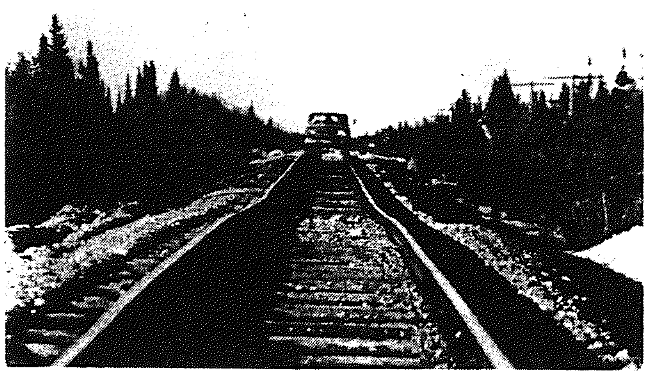

Figure 7 Typical permafrost "sinkhole" on the Hudson Bay Railway near Gillam, Manitoba

Figure 8 Experimental installation of heat pipes to arrest permafrost thaw on the Hudson Bay Railway, north of Gillam, Manitoba.

Construction of the Dempster Highway from Dawson, Yukon to Inuvik, N.W.T. during the 1970's was another milestone in northern development. This pioneering project had to address for the first time, a number of difficult problems pertaining to stability of thawing permafrost soils, particularly in the Northwest Territories north of the Richardson Mountains (Figure 9). The design philosophy, described by Huculac et al., (1978), was to preserve permafrost by building an embankment that insulates the underlying frozen soil. This meant abandoning the highway engineers' normal balanced cut and fill design methods and resorting to extensive use of soil borrow material. Unfortunately, granular construction materials are scarce along most of the route, particularly north of the Richardson Mountains. In order to obtain construction materials with acceptably low moisture or ice content, deep quarries were excavated in marginal clay-shale soil. Long haul distances, unstable thawing clay fill and the short summer season made construction difficult, resulting in substantial contractor claims for extras. These claims were invariably based on the interpretation of geotechnical conditions. Notwithstanding the construction difficulties, the Dempster Highway stands out as an excellent example where southern design and construction methodology would not work on a northern project. In this case, successful techniques were developed on a "design as you go" basis.

Figure 9 The Dempster Highway constructed over ice-rich soils on the eastern slope of the Richardson Mountains, N.W.T.

RESOURCE EXPLORATION AND DEVELOPMENT

The search for arctic oil and gas provided a great impetus to Canadian cold regions geotechnology throughout the 1970's. Experience within our vast northern region would no longer be confined to a few isolated communities but would stretch over many thousands of kilometers of proposed pipeline routes. These routes covered the Mackenzie Valley, north and south Yukon, Central Keewatin, Northern Manitoba and at least four of the Arctic Islands. In all cases extensive field data was collected and an understanding of terrain factors that influence feasibility and cost of these mega-projects was developed. The geotechnology developed during the 70's provided a basis for successful design and construction of a fully buried oil pipeline during the 80's. The Norman Wells Pipeline, described by Pick et al.(1984), effectively met the challenge of construction and operation of a pipeline in permafrost. A number of concerns relative to environmental acceptability of such a project were dispelled.

With depressed oil and gas prices, mega-projects seem to be history, however, we have been left with a tremendous legacy of technological development that has found its way into every day engineering practice. The following summarizes the pertinent achievements of geotechnical engineers who worked on those projects.

a. Field Exploration Methodology

Field investigative programs necessitated development of heli-portable drilling equipment and permafrost sampling tools that were efficient for the type of terrain crossed. Geophysical survey techniques were used for the first time on a large scale to detect and map permafrost.

b. Terrain Analyses From Airphotos

Airphoto interpretation is the key element in route selection for arctic pipelines. With little or no tree cover the surficial geology stands out. Extensive terrain classification systems were developed for mapping soils between available boreholes and for prediction of the occurrence of permafrost or ground ice.

c. Geothermal Analyses

A new discipline of geothermal analyses was introduced to geotechnical engineering practice. Extensive industry funded research was undertaken by consultants to develop and verify two dimensional numerical models of the ground thermal regime, (Hwang et al, 1972). The purpose of these models is to predict thawing, freezing and ground temperature in soils adjacent to an operating pipeline. Accuracy of the numerical procedure was determined by comparison with data from operating test installations. One early test installation, shown in Figure 10, was constructed at Norman Wells by Gas Arctic Systems in 1972.

d. Settlement and Frost Heave Analyses

Pipe displacements resulting from thaw settlement or frost heave must be predicted to establish the feasibility of any below ground system. The availability of an extensive body of field data has allowed correlation of settlement upon thaw with fundamental parameters for the frozen soil. Recently developed frost heave prediction methods based on the so-called "segregation potential" approach have resulted in an improved understanding of frost heave and the necessary mitigative methods for design (Nixon 1987). Confidence limits on results of the analyses have been established by full-scale field testing.

e. Soil-Structure Interaction

Confirmation of any proposed design in permafrost has required assurance that pipe stresses or strains will not exceed limits established by applicable codes. Evolution of numerical models for pipe stress analyses has required the geotechnical engineer to develop constitutive relationships that represent frozen and thawing soils. These parameters are applied as visco-elastic "spring constants" in the numerical analyses.

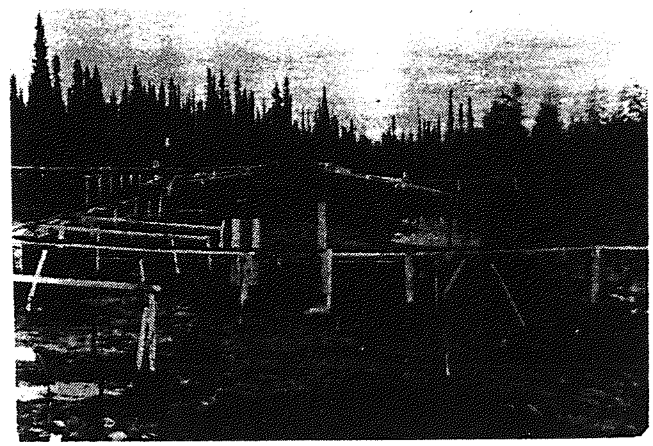

Figure 10 An early operating pipeline test loop in permafrost at Norman Wells, N.W.T.

STATUS OF CANADIAN TECHNOLOGY

Geotechnical engineering practice has evolved rapidly to meet the challenges of development in Canada's north. It has been an important element in The Berger Inquiry, Environmental Assessment and Review Panel hearings (EARP) for many projects, the Dempster Highway, Hudson Bay Railway and community development. In some cases geotechnical considerations have frequently been pivotal to the design and costing of northern projects.

Canadians are world leaders in cold regions geotechnology because it has been important to the development of our country. Some of the reasons why it remains important are:

a. It improves the quality of life for northern residents,

b. It contributes to making resource development safe, environmentally acceptable and economically sound,

c. Canadian expertise is in demand for international projects, and

d. It can improve the reliability of defense installations.

Engineers who have chosen to work in the north have had to be innovative, often gaining needed experience by learning from previous mistakes. Early initiatives at developing technology were usually directed by governments. That changed 15 years ago with intense interest in exploitation of northern resources. Industry had an immediate need to solve some of the more complex problems of operating pipelines in permafrost terrain, resulting in demand driven research by the private sector. With this phase behind us, it is time for governments to pick up some of the slack by continuing with a dedicated thrust in research and development. There is a pressing need to monitor performance of structures designed using new technology. These data are essential to understanding and refining our design techniques. The payback can be enormous when considered in relation to the maintenance costs resulting from an ill conceived design or poor construction practice.

There are no truly large-scale developments in the Canadian high arctic to date. Production of oil and gas from the Beaufort Sea is on the horizon; are we ready? Probably not quite, but with proper planning and a dedicated effort to resolve the remaining technical issues, geotechnical considerations will certainly not limit progress.

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The Future of the Canadian Geotechnical Profession

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SUMMARY

This paper forecasts the future for Canada's geotechnical engineers.

Increased opportunity and accompanying responsibility is forecast. An enhanced role, which is profit-driven, reflects maturity in project management and recognition of what the geotechnical profession offers. Perhaps the greatest opportunities for effective practice lie in large-scale natural resource developments and in waste management. They can benefit the most from effective geotechnical input.

Improved working tools are predicted for all areas of practice.

High educational standards will continue. A need for more opportunities in continuing education is identified.

Interdependence between the profession and its technical society, which is probably the strongest of any profession, will continue.

The Canadian profession leads in geotechnical technology pertinent to developing Canada. A dilemma arises from the on-off nature of Canada's developments such as the energy sector is now experiencing. If curtailments persist, practitioners who have invested in expertise in needed areas will have to seek employment elsewhere. That will mean migration to other areas of practice, and the loss of valuable talent that will be needed if and when development is re-activated.

More orderly development of Canada's resources is highly desirable.

RÉSUMÉ

Cet article prédit le futur des ingénieurs géotechniciens au Canada.

Une participation et des responsabilités accrues sont prévues. Un rôle plus important, dû à des considérations de profit, reflète une maturité en gestion des projets et une reconnaissance des apports de la profession géotechnique. La meilleure occasion d'une pratique efficace est peut-être dans l'exploitation des richesses naturelles, qui peut le plus tirer parti des apports géotechniques.

On prévoit une amélioration des outils de travail dans tous les domaines de la pratique.

Les niveaux de formation resteront élevés. On établit qu'il faudra développer l'éducation continue.

La solidarité entre la professsion et sa société technique, probablement la plus forte de toutes les disciplines, se maintiendra.

La profession a développé une expertise considérable dans diverses technologies reliées au développement du Canada. Un dilemne se présente suite à la mise en veilleuse des projets énergétiques. Si cette tendance persiste, les practiciens qui ont développé leur expertise dans ce domaine devront chercher de l'emploi ailleurs. Ceci signifie une migration vers d'autres domaines de pratique et la perte d'expériences précieuses dont on aura besoin dès que les projets seront réactivés.

Un développement plus régulier des ressources canadiennes est fortement désirable.

INTRODUCTION

The purpose of this paper is to look ahead and to forecast changes that are foreseen for the geotechnical profession in Canada. For a summary of current practice, the reader is referred to the excellent papers in the CGS Volume for the Engineering Centennial (Chapuis, Devenny; 1987). Future conditions are examined under the following headings:

- New Areas of Practice
- Changes in Project Management
- The Role of the Geotechnical Practitioner
- Employment
- Working Tools
- Improving Natural Properties
- Education
- Technology Development and Support
- Canadian Geotechnical Society

NEW AREAS OF PRACTICE

Areas of enhanced practice might be a more appropriate title for this section because the practice is not new.

Waste Management

Waste management is an area that is seeing ever increasing utilization of geotechnical professionals.

In the past geotechnical engineers played a relatively small role in waste management. Today the trend is toward multi-disciplinary teams that include specialists in geotechnical engineering, engineering geology, hydrology, hydrogeology, municipal engineering, biology, agriculture, chemistry, and even medical services. Geotechnical skills are needed to coordinate the multi-disciplinary input needed for safe efficient waste handling.

Natural Resource Development

A natural area for effective geotechnical practice is on large scale natural resource developments. These projects interact with the ground in a major way. There is a huge economic incentive to develop the projects in a way that works with the natural site conditions. The incentive is profit driven and arises from considerations that reduce cost, that reduce risk and that increase overall project reliability.

Direct Involvement in Construction

Traditional roles separate the design team from the construction team. Usually the geotechnical engineer works for the design team. Some contractors are improving their design and construction capability by employing geotechnical expertise on their team. More direct activity in construction is anticipated in the future.

Project Management

Amongst other skills an effective project manager needs to understand the interaction of disciplines working on his project. This leads to project management opportunities for geotechnical practitioners on projects that require effective input from their area.

CHANGES IN PROJECT MANAGEMENT

Some changes are anticipated in the basic approach to the development and operation of large projects. These will be identified to flag the potential impact on geotechnical practice.

Aspects that are introducing change include:

- approaches to project management
- project financing
- risk management
- approval by regulatory authorities
- timing of project approval

Approaches to Project Management

Project management practices are maturing. Managers are recognizing the need for effective input from each discipline, and steps are being taken to accommodate them.

The role of the geotechnical engineering has always been strong on projects where safety and cost considerations dictate its use. Good examples of such projects include earth dams and hydro-electric projects.

Successful application of geotechnical practice in areas such as natural resource development is rubbing off on other areas. Some development (eg. traditional planning and design in city centres) still limits effective practice. A more mature professional outlook helps.

Project Financing

Financing of a major project is an important step in project approval. Steps involved in justifying major expenditures are similar whether the financing comes from the owner or from a banking institution. Project credibility must be demonstrated. In the case of a new mine, the developer must demonstrate that geotechnical aspects have been assessed by a credible practitioner, that risks have been identified and taken into account, and that the development is feasible.

Risk Management

There are exciting new aspects of risk management on large projects. Some projects are so large that failure to perform can put the owner or owners out of business. To offset this risk, owners may carry insurance that protects them against production interruptions. Claims arising from the latter can amount to hundreds of millions of dollars. If invoked, the insurance companies abandon their traditional arm's length role and delve into all aspects of the project; including the basic feasibility assessment and the basic design assumptions. Needless to say, the original assessment had better be sound, or the insurance company may have grounds to deny the claim.

Another aspect of risk management involves planning each aspect of the project to reduce or manage risk. Frequently, there are options available that allow this. On a natural resource development project, that means effectively involving geotechnical assessment in every step of the project.

The risk management aspect creates opportunities for geotechnical involvement in senior management.

Approval by Regulatory Authorities

Approval by regulatory authorities is necessary before a project can proceed. Normally, the owner should impose more rigid standards than the regulatory authorities. However, effective geotechnical input is necessary if the project is to be approved. This is especially true for projects that intereact with the natural conditions in a major way - as do mega projects and natural resource development projects.

Timing of Project Approval

Effective geotechnical site exploration, evaluation and input to project planning require time and money. The work is most effective if carried out in several phases.

Many large projects require government approval before work can get underway. A dilemma occurs when the approval is so late that exploration is delayed. Without approval exploration expenses cannot be justified. After approval there may not be enough time to explore and input to project planning effectively. Developments frequently proceed on the basis of very limited site information.

Hopefully, in future, staged approvals will be given so that exploration can proceed in an efficient, effective manner.

The preceeding changes offer an enhanced role for the geotechnical practitioner. The role is enhanced by a more mature outlook on the part of the other professionals. That mature outlook is also reinforced by liability insurance considerations that limit practice to areas of personal expertise.

ROLE OF THE GEOTECHNICAL PRACTITIONER

Basic services offered by the geotechnical profession today will still be needed in the future. These include exploring and interpreting ground conditions relevant to:

- site selection
- site development and best use
- the most appropriate approach to site development, design, mining and construction
- monitoring and remedial works
- reclamation and restoration

The importance of effective geotechnical input to major projects is receiving increasing recognition from project owners, from risk managers and from regulatory authorities. This recognition will enhance the role, and the responsibility, of the geotechnical practitioner.

In order to provide the input required, the geotechnical practitioner will have to be intimately involved in project design and operations. There is also a role in senior project management.

A full description of the complete geotechnical involvement in a project includes:

- a full understanding of the final results desired.
- a full understanding of the options for design development and operations.
- interaction with the project team to understand alternatives and the consequences and risk associated with each.
- work with the project team to identify the optimum approach to the project, taking risk into account.
- providing effective input to:
  - ° guidelines for project development and design
  - ° construction plans and specifications
  - ° operating guidelines
- having an effective role during construction to:
  - ° verify that ground conditions exposed are the same as were assumed in design
  - ° monitor construction practice to see that it corresponds with the design intent
  - ° monitor interaction between construction procedures and ground conditions to verify design assumptions
  - ° if deviations are identified, assess the impact and take corrective action
- having an effective role during operations to:
  - ° verify performance
  - ° repeat tasks noted for construction
  - ° fine-tune operations as appropriate

Work in a multi-disciplinary team environment is common. This requires effective input from mature professionals. It is desirable to have a knowledgeable geotechnical presence involved in senior management of projects that can benefit from effective geotechnical input.

EMPLOYMENT

Type of Employer

Geotechnical engineers will still find employment through the same three types of employer:

- Consulting: This will remain as the main employer. From the consulting base, practitioners will serve a wide variety of clients and projects.

  Consulting practice will probably remain as the best training ground for the young professional. It also offers mature professionals an opportunity to join forces or to work alone.

- Industry: This category includes government agencies with specific mandates for project development such as hydro electric projects.

  Employment opportunities in this area will probably increase as individual projects acquire their own resident staff.

  Exposure through employment by industry tends to be different from consulting. Assignments to a specific project tend to be longer and to involve other duties.

  Increased direct utilization of geotechnical expertise on the contractor's side of construction activity is forecast. Direct employment by foundation contractors is another growth area for geotechnical engineers.

  Opportunities for a senior role in project management will probably be greatest on specific industry projects.

- Government/Regulatory: There will be a continuing need for knowledgeable professionals in regulatory agencies.

Type of Employment

Historically, engineers have worked as employees, although many consulting firms are employee-owned.

In the last few years, consultants and industry have found it expensive to reduce staff during slack times. As a result, they are reluctant to hire new employees. Term contracts are offered instead.

Term contracts are likely to become more common in the future. They will be a common form of employment on major projects. Consultants may be seconded to specific projects on a contract basis.

Liability Insurance

Traditionally, liability insurance coverage has been provided by the employer.

On a major project, the project carries the full insurance needed. The liability of each contributing consultant is often limited.

We expect to see more consolidation of the design/construction team under one project insurance policy in the future. On high risk projects, this can even involve formal progress reviews by representatives of the insurance companies.

Practitioners with term employment will have to ensure that they have adequate insurance coverage.

Location of Employment

Geotechnical engineers tend to work in design centres so they can be near their clients. There is also a field component that must take place on the site of each development.

There is no replacement for face to face meetings with the design team. However, with today's improvements in communications between computers, it is possible for a design team to be dispersed across the country. This will alleviate, but not eliminate, the need for practitioners to work in design centres.

Improvements in communications will improve the effectiveness of on-site work.

Alberta currently has the highest per capita concentration of geotechnical engineers in Canada. Much of their employment has been associated with the energy industry.

With the curtailment of new energy developments, the demand for Alberta's geotechnical engineers has diminished. Unless steps are taken to retain key skills, they will move to other areas of employment.

WORKING TOOLS

Exploration

Exploration is the key to determining site conditions. Consequently, exploration is one of the most important aspects of geotechnical engineering. It is also potentially one of the weakest.

Our ability to analyse or to predict performance is much better than our ability to determine the subsurface conditions that control that performance.

In future, we can expect to see improvements in site exploration tools and methods. Some improvements expected in the near term include:

1) Surface Geophysical Exploration - greater use can be made of surface geophysical methods to supplement information obtained by conventional methods. Examples:
   - seismic
   - sonar/radar imagery
   - electrical methods: resistivity, induction, and transient

2) Down-hole Geophysical Logging - techniques used in the oil industry will be adopted to the geotechnical investigation environment to provide direct information on:
   - formation density
   - water content
   - clay content
   - clay minerology
   - attitude of dipping beds
   - pore water pressures

3) In-Situ Testing - a better understanding and application of in-situ testing techniques is highly desirable. Calibration to show how the various methods relate to traditional methods will help.

Laboratory Testing

Laboratory equipment and testing procedures are getting more specialized and more sophisticated. Through automation, the routine analyses are also being performed more efficiently.

We are already observing a consolidation of specialized test equipment in a few labs. We can expect to see more of this.

Computer monitoring and data reduction, already in use in many laboratories, will become more common.

Analysis

Tools to analyse subsurface conditions and to predict response to various types of loading abound.

We can expect more widespread use of sophisticated tools as computing power becomes cheaper and as the programs become more user friendly.

Ongoing field monitoring is part of the safety program on some projects. Automated systems are evolving that allow transmission of data from the field units to central facilities where it can be reduced on a timely basis. The next step will be programs to automatically interpret data from a specific project.

Communicating Geological Information

More effective use of a geologist's interpretation is highly desirable. Unfortunately, the geologist's ability to communicate his interpretations in a form that others can use is limited. Oil sands projects have developed solutions that offer assistance in this area.

The geological data base of an oil sand project is very large - involving millions of pieces of data. Computerized filing systems have been developed to store the data and to help the geologist sort through it. The geologist's best interpretation is then summarized in a three dimensional computerized block model that is available for other users such as mine planners or project designers. This allows the geologist to communicate more effectively.

We hope to see this communication system expanded to other areas.

Follow-up

Follow-up to confirm predictions made in the early stage of a project is an important part of geotechnical engineering. Follow-up may be to verify subsurface conditions, or to assess the affect of construction operations, or to monitor performance of an engineering structure.

Opportunities for geotechnical follow-up have been limited in conventional design teams. More opportunities are anticipated in the future when the importance to projects is realized.

Library

An extensive library is an important part of any geotechnical practice. Information includes books, periodicals, and case histories.

With today's computer assistance, even greater library access is possible. Computer-aided search routines will improve efficiency.

Regional data bases were developed for some areas 20 years ago. The data has not enjoyed widespread use. With computer assistance, the data should become much more accessible and hence usable.

Computers

The geotechnical engineer will not be replaced by a computer. The task of interpreting subsurface conditions and of assessing the interaction with site development is too compex. However, the ability of the geotechnical engineer to make his assessments will be greatly enhanced with the use of computers.

There has been a quiet revolution in geotechnical work as computers have been applied to every area of practice. We can expect to see continuing change, due to even greater use of computers.

Increased use of computers will leave more time for thinking about the project.

IMPROVING NATURAL PROPERTIES

Usually, the geotechnical engineer accepts natural site materials and develops plans to work with them. However, there are methods of upgrading material properties. These offer new opportunities and responsibilities for the geotechnical engineer. Some of these methods are noted below.

1) Replacement - undesirable materials can be excavated and replaced by more desirable materials. This is common practice in road-building and in landfill applications.

2) In-Situ Improvement - weak materials are strengthened by making them denser and stronger. Methods used are:
   - surcharging to induce consolidation and stability under design loads
   - dynamic consolidation of natural materials using mechanical methods or blasting
   - using electro-osmosis to enhance drainage and consolidation of weak materials.

3) Reinforcement - reinforcement can be installed to strengthen weak materials:
   - sand or stone columns
   - geotextiles - to provide strength, to enhance drainage, and to resist erosion.

4) Chemical Strengthening - improve soil strength through the application of cementing agents, electro-osmosis or heat. In northern areas freezing can be induced and maintained to provide strength.

5) Chemical Alteration - chemical treatment to improve properties of natural or waste materials. Examples of desired properties are:
   - chemically attractive instead of dispersed clays
   - resist erosion
   - support instead of retard vegetation.

As land becomes more valuable, and as practitioners understand the full potential of what can be done, we can expect to see greater application of programs to improve properties.

In the more distant future developments in the nuclear industry may result in low cost energy. Cheap energy would make it possible to improve natural conditions by heating or even melting to create glass or ceramic foundations. A whole new area of materials science will evolve to deal with this aspect.

EDUCATION

Geotechnical practice requires considerable background knowledge and practical field experience. Advanced education is highly desirable. Consequently, 80% of Canadian geotechnical practitioners have master's degrees, and 20% have PhD degrees.

Formal training is usually obtained in the early years. Many Canadian universities offer excellent courses in geotechnical engineering.

Most graduate research is conducted inside the universities. Short-term studies that are suitable for research in a university environment have been well researched. However, there are many practical field problems begging for attention. Hopefully, the future will see more emphasis on practical field research involving joint efforts by industry and the universities (Devenny, 1986).

Subsequent training and upgrading is obtained through the following sources:

- On-the-job experience
- Technical literature:
  - Canadian Geotechnical Journal
  - Geotechnical News
  - CGS technical publications
  - Other international publications
- University short courses
- Specialty seminars and symposia
- Local CGS meetings

There is a need for more continuing education opportunities for the seasoned practitioner.

In the near future, the Engineering Institute of Canada will be awarding Continuing Education Units or points for attendance at approved courses or learning experiences. This system will keep track of all earned continuing education.

The author is not aware of formal courses that deal with:

- Geotechnically oriented risk management for large projects
- Effective geotechnical input to major projects
- Advanced project management specifically for geotechnical engineers

The subjects noted may be beyond the experience of those in university, but they need to be given exposure.

THE CANADIAN GEOTECHNICAL SOCIETY

The Canadian Geotechnical Society was created by the Canadian geotechnical profession to serve its unique needs.

Needs and how they are served are noted below:

- An opportunity to learn about new techniques and case histories:
  - local meetings
  - annual meetings
  - specialty symposia
  - Canadian Geotechnical Journal
- A communication and news service:
  - Geotechnical News
- Linkage to national and international societies:
  - EIC
  - Canadian Geoscience Council
  - ISSMFE
  - IAEG
  - ISRM
- A spokesman on national issues:
  - CGS

The Canadian Geotechnical Society meets many of the needs of the profession today. Efforts are underway to make the service even better. The relationship between the Society and the Canadian Geotechnical profession should grow even stronger in the future.

We can expect to see closer ties develop between The Canadian Geotechnical Society and societies representing associated professions such as:

- civil engineers
- mining engineers
- hydrogeologists
- environmentalists
- contractors

TECHNOLOGY DEVELOPMENT AND SUPPORT

Technology Development

The geotechnical profession has been very adept at developing solutions to unique challenges involved in developing Canada. Much of the technology has been developed for use on specific projects. Example areas of unique geotechnical expertise are:

- design of mine waste facilities in mountainous terrain
- oil sands developments:
  - structural foundations
  - mining operations
  - waste management
- frontier energy development:
  - offshore exploration and production facilities
  - iceberg prediction and protection
  - sea ice - resisting applied forces and using it for work platforms
  - pipelines
- northern development:
  - permafrost engineering

Individual projects have funded much of the new technology needed. Commonly, the expertise has been obtained from the consulting community.

As a result of these developments, Canada enjoys world leadership in the key areas needed to develop its natural resources.

Retention of Expertise

Canada has developed considerable expertise in areas peculiar to Canada's developments. In order to attract and hold the highly motivated individuals who develop expertise in needed areas there must be continuity. If there is no continuity the individuals will seek employment in new aeas, and their expertise will be lost.

The problem of retaining expertise is illustrated by the efect of the recent drop in oil prices. The price drop has delayed development of energy supplies from frontier areas. This has greatly reduced the demand for expertise in those areas. If the curtailment persists the expertise, and with it Canada's ability to develop its resources will be lost.

CLOSURE

An exciting future with an enhanced role and enhanced responsibility is forecast.

The enhanced role for geotechnical practice is profit-driven. It reflects maturity on the part of the project managers, and recognition of what geotechnical engineering can contribute.

Improvements are forecast in all areas of practice.

High educational standards will persist. A need for more continuing education opportunities is identified.

The profession and its technical society, the Canadian Geotechnical Society, are quite dependent on one another. The interdependence will probably grow in future.

The Canadian geotechnical community excels in techniques needed to develop Canada. Considerable expertise has been created for specific development in Canada. Problems arise from the on-off nature of development plans. If the deferral persists, practitioners who have invested in expertise in that area will have to seek employment elsewhere. That will mean migration to other areas of practice. Ultimately, it will mean a loss of skills needed to develop Canada's resources.

More orderly development, or temporary assignment to programs aimed at reducing development costs, is advocated.

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