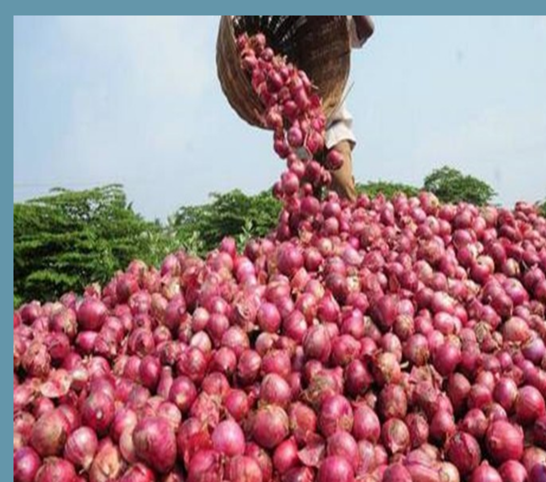
SSIGL 6

NATIONAL GUIDELINES

For Small Scale Irrigation Development in Ethiopia

Geology and Engineering Geology Study



November 2018

Addis Ababa

MINISTRY OF AGRICULTURE

National Guidelines for Small Scale Irrigation Development in Ethiopia

SSIGL 6: Geology and Engineering Geology Study

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National Guidelines for Small Scale Irrigation Development in Ethiopia
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DISCLAIMER

Ministry of Agriculture through the Consultant and core reviewers from all relevant stakeholders included the information to provide the contemporary approach about the subject matter. The information contained in the guidelines is obtained from sources believed tested and reliable and are augmented based on practical experiences. While it is believed that the guideline is enriched with professional advice, for it to be successful, needs services of competent professionals from all respective disciplines. It is believed, the guidelines presented herein are sound and to the expected standard. However, we hereby disclaim any liability, loss or risk taken by individuals, groups, or organization who does not act on the information contained herein as appropriate to the specific SSI site condition.

FORWARD

Ministry of Agriculture, based on the national strategic directions is striving to meet its commitments in which modernizing agriculture is on top of its highest priorities to sustain the rapid, broad-based and fair economic growth and development of the country. To date, major efforts have been made to remodel several important strategies and national guidelines by its major programs and projects.

While efforts have been made to create access to irrigation water and promoting sustainable irrigation development, several barriers are still hindering the implementation process and the performance of the schemes. The major technical constrains starts from poor planning and identification, study, design, construction, operation, and maintenance. One of the main reasons behind this outstanding challenge, in addition to the capacity limitations, is that SSIPs have been studied and designed using many ad-hoc procedures and technical guidelines developed by various local and international institutions.

Despite having several guidelines and manuals developed by different entities such as MoA (IDD)-1986, ESRDF-1997, MoWIE-2002 and JICA/OIDA-2014, still the irrigation professionals follow their own public sources and expertise to fill some important gaps. A number of disparities, constraints and outstanding issues in the study and design procedures, criteria and assumptions have been causing huge variations in all vital aspects of SSI study, design and implementation from region to region and among professionals within the same region and institutions due mainly to the lack of agreed standard technical guidelines. Hence, the SSI Directorate with AGP financial support, led by Generation consultant (GIRDC) and with active involvement of national and regional stakeholders and international development partners, these new and comprehensive national guidelines have been developed.

The SSID guidelines have been developed by addressing all key features in a comprehensive and participatory manner at all levels. The guidelines are believed to be responsive to the prevalent study and design contentious issues; and efforts have been made to make the guidelines simple, flexible and adaptable to almost all regional contexts including concerned partner institution interests. The outlines of the guidelines cover all aspects of irrigation development including project initiation, planning, organizations, site identification and prioritization, feasibility studies and detail designs, contract administration and management, scheme operation, maintenance and management.

Enforceability, standardization, social and environmental safeguard mechanisms are well mainstreamed in the guidelines, hence they shall be used as a guiding framework for engineers and other experts engaged in all SSI development phases. The views and actual procedures of all relevant diverse government bodies, research and higher learning institutions, private companies and development partners has been immensely and thoroughly considered to ensure that all stakeholders are aligned and can work together towards a common goal. Appropriately, the guidelines will be familiarized to the entire stakeholders working in the irrigation development. Besides, significant number of experts in the corresponding subject matter will be effectively trained nationwide; and the guidelines will be tested practically on actual new and developing projects for due consideration of possible improvement. Hence, hereinafter, all involved stakeholders including government & non-governmental organizations, development partners, enterprises, institutions, consultants and individuals in Ethiopia have to adhere to these comprehensive national guidelines in all cases and at all level whilst if any overlooked components are found, it should be documented and communicated to MOA to bring them up-to-date.

Therefore, I congratulate all parties involved in the success of this effort, and urge partners and stakeholders to show a similar level of engagement in the implementation and stick to the guidelines over the coming years.

H.E. Dr. Kaba Urgessa
State Minister, Ministry of Agriculture

SMALL SCALE IRRIGATION DEVELOPMENT VISION

Transforming agricultural production from its dependence on rain-fed practices by creating reliable irrigation system in which smallholder farmers have access to at least one option of water source to increase production and productivity as well as enhance resilience to climate change and thereby ensure food security, maintain increasing income and sustain economic growth.

ACKNOWLEDGEMENTS

The preparation of SSIGLs required extensive inputs from all stakeholders and development partners. Accordingly many professionals from government and development partners have contributed to the realization of the guidelines. To this end MOA would like to extend sincere acknowledgement to all institutions and individuals who have been involved in the review of these SSIGLs for their comprehensive participation, invaluable inputs and encouragement to the completion of the guidelines. There are just too many collaborators involved to name exhaustively and congratulate individually, as many experts from Federal, regional states and development partners have been involved in one way or another in the preparation of the guidelines. The contribution of all of them who actively involved in the development of these SSIGLs is gratefully acknowledged. The Ministry believes that their contributions will be truly appreciated by the users for many years to come.

The Ministry would like to extend its appreciation and gratitude to the following contributors:

- Agriculture Growth Program (AGP) of the MoA for financing the development and publication of the guidelines.
- The National Agriculture Water Management Platform (NAWMP) for overseeing, guidance and playing key supervisory and quality control roles in the overall preparation process and for the devotion of its members in reviewing and providing invaluable technical inputs to enrich the guidelines.
- Federal Government and Regional States organizations and their staff for their untiring effort in reviewing the guidelines and providing constructive suggestions, recommendations and comments.
- National and international development partners for their unreserved efforts in reviewing the guidelines and providing constructive comments which invaluably improved the quality of the guidelines.
- Small-scale and Micro Irrigation Support Project (SMIS) and its team for making all efforts to have quality GLs developed as envisioned by the Ministry.

The MOA would also like to extend its high gratitude and sincere thanks to AGP's multi development partners including the International Development Association (IDA)/World Bank, the Canada Department of Foreign Affairs, Trade and Development (DFATD), the United States Agency for International Development (USAID), the Netherlands, the European Commission (EC), the Spanish Agency for International Development (AECID), the Global Agriculture and Food Security Program (GAFSP), the Italy International Development Cooperation, the Food and Agriculture Organization (FAO) and the United Nations Development Program (UNDP).

Moreover, the Ministry would like to express its gratitude to Generation Integrated Rural Development Consultant (GIRDC) and its staff whose determined efforts to the development of these SSIGLs have been invaluable. GIRDC and its team drafted and finalized all the contents of the SSIGLs as per stakeholder suggestions, recommendations and concerns. The MoA recognizes the patience, diligence, tireless, extensive and selfless dedication of the GIRDC and its staff who made this assignment possible.

Finally, we owe courtesy to all national and International source materials cited and referred but unintentionally not cited.

Ministry of Agriculture

DEDICATIONS

The National Guidelines for Small Scale Irrigation Development are dedicated to Ethiopian smallholder farmers, agro-pastoralists, pastoralists, to equip them with appropriate irrigation technology as we envision them empowered and transformed.

LIST OF GUIDELINES

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ACRONYMS

AGP	Agricultural Growth Program
BH	Borehole
D/s	Down Stream
DCP	Dynamic cone penetration
GFR	Geological Factors Rating
GIRDC	Generation Integrated Rural Development Consultant
GIS	Geographic Information System
GPS	Global Positioning System
GWT	Ground Water Table
ISRM	International Society for Rock Mechanics
MOANR	Ministry of Agriculture and Natural Resource
MOWIE	Ministry of Water, Irrigation and Electricity
MoWR	Ministry of Water Resource
ORN	Overseas Road Note
RMR	Rock Mass Rating
RQD	Rock Quality Designation
SCR	Solid core recovery
SPT	Standard Penetration Test
SSID	Small Scale Irrigation Development
SSIGL	Small Scale Irrigation Guideline
SSIP	Small Scale Irrigation Project
SSIS	Small Scale Irrigation Scheme
TCR	Total core recovery
TP	Turning Point
TRRL	Transport Road Research Laboratory
U/s	Up stream
UK	United Kingdom
USCS	Unified Soil Classification System
VES	Vertical Electrical Sounding

PREFACE

While irrigation development is at the top of the government's priority agendas as it is key to boost production and improve food security as well as to provide inputs for industrial development. Accordingly, irrigated land in different scales has been aggressively expanding from time to time. To this end, to enhance quality delivery of small-scale irrigation development planning, implementation and management, it has been decided to develop standard SSI guidelines that must be nationally applied. In September 2017 the Ministry of Agriculture (MoA) had entrusted Generation Integrated Rural Development Consultant (GIRDC) to prepare the National Small-scale Irrigation Development Guidelines (SSIGLs).

Preparation of the SSIGLs for enhancing development of irrigated agriculture is recognized as one of the many core initiatives of the MoA to improve its delivery system and achieve the targets in irrigated agriculture and fulfill its mission for improving agricultural productivity and production. The core objective of developing SSIGLs is to summarize present thinking, knowledge and practices to enable irrigation practitioners to properly plan, implement and manage community managed SSI schemes to develop the full irrigation potential in a sustainable manner.

As the SSIGLs are prepared based on national and international knowledge, experiences and practices, and describe current and recommended practice and set out the national standard guides and procedures for SSI development, they serve as a source of information and provide guidance. Hence, it is believed that the SSIGLs will contribute to ensuring the quality and timely delivery, operation and maintenance of SSI schemes in the country. The SSIGLs attempt to explain and illustrate the important concepts, considerations and procedures in SSI planning, implementation and management; and shall be used as a guiding framework for professionals engaged in SSI development. Illustrative examples from within the country have been added to enable the users understand the contents, methodologies presented in the SSIGLs.

The intended audiences of the SSIGLs are government organizations, NGOs, CSOs and the private sector involved in SSI development. Professionally, the SSIGLs will be beneficial for experienced and junior planners, experts, contractors, consultants, suppliers, investors, operators and managers of SSI schemes. The SSIGLs will also serve as a useful reference for academia and researchers involved and interested in SSI development. The SSIGLs will guide to ensure that; planning, implementation and management of SSI projects is formalized and set procedures and processes to be followed. As the SSIGLs provide information and guides they must be always fully considered and applied by adapting them to the local specific requirements.

In cognizance with the need for quality SSIGLs, the MoA has duly considered quality assurance and control during preparation of the guidelines. Accordingly, the outlines, contents and scope of the SSIGLs were thoroughly discussed, reviewed and modified by NAWMP members (senior professionals from public, national and international stakeholder) with key stakeholders in many consultative meetings and workshops. Moreover, at each milestone of SSIGL preparation, resource persons from all stakeholders reviewed and confirmed that SSIGLs have met the demands and expectations of users.

Moreover, the Ministry has mobilized resource persons from key Federal, National Regional States level stakeholders and international development partners for review, validation and endorsement of the SSIGLs.

Several hundreds of experienced professionals (who are very qualified experts in their respective fields) from government institutions, relevant private sector and international development partners have significantly contributed to the preparation of the SSIGLs. They have been involved in all aspects of the development of SSIGLs throughout the preparation process. The preparation process included a number of consultation meetings and workshops: (i) workshop to review inception report, (ii) workshop on findings of review of existing guidelines/manuals and proposed contents of the SSIGLs, (iii) meetings to review zero draft SSI GLs, (iv) review workshop on draft SSI GLs, (v) small group review meetings on thematic areas, (vi) small group consultation meetings on its final presentation of contents and layout, (vii) consultation mini-workshops in the National States on semi-final versions of the SSIGLs, and (viii) final write-shop for the appraisal and approval of the final versions of SSIGLs.

The deliberations, concerns, suggestions and comments received from professionals have been duly considered and incorporated by the GIRD Consultant in the final SSIGLs.

There are 34 separate guidelines which are categorized into the following five parts concurrent to SSI development phases:

Part-I. Project Initiation, Planning and Organization Guideline which deals with key considerations and procedures on planning and organization of SSI development projects.

Part-II. Site Identification and Prioritization Guideline which treats physical potential identification and prioritization of investment projects. It presents SSI site selection process and prioritization criteria.

Part-III. Feasibility Study and Detail Design Guidelines for SSID dealing with feasibility study and design concepts, approaches, considerations, requirements and procedures in the study and design of SSI systems.

Part-IV. Contract Administration and Construction Management Guidelines for SSI development presents the considerations, requirements, and procedures involved in construction of works, construction supervision and contract administration.

Part-V. SSI Scheme Management, Operation and Maintenance Guidelines which covers SSI Scheme management and operation.

Moreover, Tools for Small Scale Irrigation development are also prepared as part of SSIGLs.

It is strongly believed and expected that; the SSIGLs will be quickly applied by all stakeholders involved in SSI development and others as appropriate following the dissemination and familiarization process of the guidelines in order to ensure efficient, productive and sustainable irrigation development.

The SSIGLs are envisioned to be updated by incorporating new technologies and experiences including research findings. Therefore, any suggestions, concerns, recommendations and comments on the SSIGLs are highly appreciated and welcome for future updates as per the attached format below. Furthermore, despite efforts in making all types of editorial works, there may still errors, which similarly shall be handled in future undated versions.

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UPDATING AND REVISIONS OF GUIDELINES

The GLs are intended as an up-to-date or a live document enabling revisions, to be updated periodically to incorporate improvements, when and where necessary; may be due to evolving demands, technological changes and changing policies, and regulatory frameworks. Planning, study and design of SSI development interventions is a dynamic process. Advancements in these aspects are necessary to cope up with the changing environment and advancing techniques. Also, based on observation feedbacks and experiences gained during application and implementation of the guidelines, there might be a need to update the requirements, provisions and procedures, as appropriate. Besides, day-by-day, water is becoming more and more valuable. Hence, for efficient water development, utilization and management will have to be designed, planned and constructed with a new set up of mind to keep pace with the changing needs of the time. It may, therefore, be necessary to take up the work of further revision of these GLs.

This current version of the GLs has particular reference to the prevailing conditions in Ethiopia and reflects the experience gained through activities within the sub-sector during subsequent years. This is the first version of the SSI development GLs. This version shall be used as a starting point for future update, revision and improvement. Future updating and revisions to the GLs are anticipated as part of the process of strengthening the standards for planning, study, design, construction, operation and management SSI development in the country.

Completion of the review and updating of the GLs shall be undertaken in close consultation with the federal and regional irrigation institutions and other stakeholders in the irrigation sub-sector including the contracting and consulting industry.

In summary, significant changes to criteria, procedures or any other relevant issues related to technological changes, new policies or revised laws should be incorporated into the GLs from their date of effectiveness. Other minor changes that will not significantly affect the whole nature of the GLs may be accumulated and made periodically. When changes are made and approved, new page(s) incorporating the revision, together with the revision date, will be issued and inserted into the relevant GL section.

All suggestions to improve the GLs should be made in accordance with the following procedures:

I. Users of the GLs must register on the MOA website: Website: www.moa.gov.et
II. Proposed changes should be outlined on the GLs Change Form and forwarded with a covering letter or email of its need and purpose to the Ministry.
III. Agreed changes will be approved by the Ministry on recommendation from the Small-scale Irrigation Directorate and/or other responsible government body.
IV. The release date of the new version will be notified to all registered users and authorities.

Users are kindly requested to present their concerns, suggestions, recommendations and comments for future updates including any omissions and/or obvious errors by completing the following revisions form and submitting it to the Ministry. The Ministry shall appraise such requests for revision and will determine if an update to the guide is justified and necessary; and when such updates will be published. Revisions may take the form of replacement or additional pages. Upon receipt, revision pages are to be incorporated in the GLs and all superseded pages removed.

Suggested Revisions Request Form (Official Letter or Email)

To: ---
From: ---
Date: --
Description of suggested updates/changes: Include GL code and title, section title and # (heading/subheading #), and page #.

GL Code and Title	Date	Sections/ Heading/Subheading/ Pages/Table/Figure	Explanation	Comments (proposed change)

Note that be specific and include suggested language if possible and include additional sheets for comments, reference materials, charts or graphics.

GLs Change Action

Suggested Change	Recommended Action	Authorized by	Date

Director for SSI Directorate**: _________________________Date: _________________**

The following table helps to track initial issuance of the guidelines and subsequent Updates/Versions and Revisions (Registration of Amendments/Updates).

Revision Register

Version/Issue/Revision No	Reference/Revised Sections/Pages/topics	Description of revision (Comments)	Authorized by	Date

1 SCOPE AND OBJECTIVE OF THE GUIDELINE

The objective of this guideline is to standardize the geological and geotechnical investigation practice, methods and procedures to meet internationally accepted qualities for foundation and construction materials investigations for micro-dams, Diversion weirs, simple intakes, crossing and I & D hydraulic structures associated with Small Scale Irrigation Projects (SSIPs) in Ethiopia. It is intended to have independent guideline for SSIPs by filling the gaps of the previous guidelines prepared by different bodies for irrigation projects of different magnitudes/scales

The scope of this guideline is to outline the desirable quantity and quality of geological and geotechnical investigations required for safe and economical design and construction of SSIPs in Ethiopia.

In this regard, geological and engineering geological mapping give first information about soil and rock material around the site and information concerning geological structures and geomorphological features. Mapping aspects and recommended scales are shown. Mass movement (landslide) is known in its hazardous nature hence, in any hydraulic structures stability condition has to be checked for the safety of the structure. Identification, recognition, type of landslides and their possible remedial measures are covered.

Water tightness in any hydraulic structures has to be checked using field permeability tests, accordingly constant head, variable head, packer tests and pumping out tests are treated in the guideline. In addition general geology of Ethiopia, seismcity, groundwater observations and exploration conditions are presented for quick use in any interest area. Furthermore symbols for rock, soil and other geological features are given in the guideline in order to have consistency in use of symbols in different SSIPs. Other site investigation methods like geophysical methods, drilling, test pitting, augering and other excavation methods along with sampling and sample handling are treated .

This guideline describes the geological and geotechnical parameters which require investigative data, information and design recommendations. It also explains the different geological and geotechnical parameters that need to be addressed in different levels/stages of SSIPs projects. It provides information and guidance on parameters to be studied; and methods of collecting geological and geotechnical data with respect to these parameters. Checklists to be dealt with in the geological and geotechnical investigations; and procedures of obtaining data and information accompanied with outlines of presenting the information are included.

Laboratory testing, classification and characterization procedures are also discussed in the guideline. Geotechnical analysis using produced data and evaluations of the results with respect to national and international standards, requirements and specifications are presented.It is fully recognized, however, that the specific investigation plan and studies for individual projects cannot realistically be standardized, and will vary widely according to site conditions, type of hydraulic structure, hazard classification, and design phase.

2 GEOLOGICAL AND GEOTECHNICAL INVESTIGATION OF DIVERSION PROJECT SITE

2.1 PLANNING

Geological and Geotechnical study for Diversion project structures planned based on the data and informations gathered from desk studies and site reconnaissance visit.

2.1.1 Desk studies (office work)

The Geotechnical Engineering study expert (team) should become completely familiar with the proposed project elements by studying the preliminary plans provided by the Designer of head work. Location and size of structures and area coverage should be determined.

Initial site review begins by identifying the geologic processes that have influenced the environment of the project site.

The goal of the office review is to plan site reconnaissance and prepare a conceptual plan for surface and subsurface exploration. The following maps and informations could be collected from the appropriate organizations and used during the desk studies.

a. Topographic Maps (could be obtained from Ethiopian Map Agency)
b. Maps & reports (Geolgical, hydrogeological etc, could br obtained from Geological survey of Ethiopia)
c. Aerial Photos and satellite image (could be obtained from Ethiopian Map Agency)
d. Previous Site Exploration Data and site use
e. Construction Records (as built), if there exist, around vacinity of the project

2.1.2 Short site visit

The field short site visit should be done with the preliminary plans developed based the desk study. Cross sections provided with the preliminary plans should be checked at the field.

Location, type and depth of any existing or abandoned structures and/or foundations and location, size, and condition of anyrock outcrops with respect to the proposed structure have to be noted. Inspect tentative structure lay out and the stream banks upstream and downstream for evidence of sliding, scour and general stability and avaability proximity & accessibility of construction materials.

Compare the topography of the site with that shown on maps and try to confirm the assumptions made during the office review concerning the site geology. Observe and note natural occurring exposures evident at river banks, natural escarpments, quarries, and rock outcrops. Measure the direction of any observed geological structure features.

Photographs are valuable records of the sitevisit and should be labeled with the approximate stationing, direction of view, date, and abrief title. Photos should be obtained of all the site features listed above and of the probable exploration locations.

2.1.3 Preparation of geological and geotechnical investigation plan

The short site visit and office review results are generally used to develop the geotechnical designs required either in the feasibility or/and Design Phase.

The goal of the engineering geological or geotechnical investigation program is to obtain the engineering properties of the soil or rock and to define the lateral extent, elevation, and thickness of each identifiable soil/rock stratum, within a depth that could affect the design requirement.
The type, location, size and depth of the explorations and testing are dependent upon the nature and size of the project and on the degree of complexity and critical nature of the subsurface conditions.

Planning for the investigation of diversion project site
Subsurface explorations for Diverssion project site may include excavation of test pits and associated insitu tests, geophysical survey and some time drilling of short boreholes with the necessary insitu tests. The test pits are simply manually or mechanically dug holes, often large enough for persons to work in, used to investigate subsurface strata, determine ground water conditions, or sample granular material sources. Small test pits may be used as percolation test pits or holes.

Planning for investigation using test pits excavation may include how to manage the following acticities.

- General Sampling Requirements
- The need, use possibility of collecting undisturbed Sampling
- Selecting and conducting of relivant type of In-Situ Testings as per the site condition
- Evaluation of Groundwater Conditions in line with the influence on the structures (Diversion structures, canal and or pond).

Geophysical survey are especially useful for defining lateral geologic stratigraphy, and can be useful to identify buried erosion channels, detailed rock surface location, over all rock quality.

Geophysical testing could be used in combination with information from direct methods of exploration, such as SPT, DCP, etc. to establish stratification of the subsurface materials, the profile of the top of bedrock and bedrock quality, depth to groundwater, limits of types of soil deposits, the presence of voids, anomalous deposits, buried pipes, and depths of existing foundations.

There are a number of different geophysical in-situ tests that can be used for stratigraphic information and determination of engineering properties. Imaging method of Vertical Electrical Sounding (VES) and seismic refraction using a hammer are sufficient to characterize the foundations of diversion structures and other associated structures.

Sites characterized by complex geological formations and thick alluvial deposite may need investigation through drilling short explatory boreholes. Detail panning pproceedures for investigation using exploratory borehole drilling presented in the sub topic 3.1.3.4

2.2 GEOLOGICAL MAPPING OF DIVERSION STRUCTURES SITE

2.2.1 General

A map showing the distribution of rock units and geological structures across a given area, usually on a plane surface, is thus a geological map. Geological field mapping is the process of selecting an area of interest and identifying all the geological aspects of that area with the purpose of preparing a detailed geological report and a map to summarize the report. A geological map will thus show the various rock types of the area, the structures, geological formations, age relationships, distribution of mineral ore deposits and fossils etc. and all these features may be super imposed over a topographic map or a base map. The amount of detail shown in a map depends largely on the scale and a smaller scale will naturally disclose finer detail. Basically, the quality of a geologic map will depend upon the accuracy and the precision of the field work. Further, still quality depends on the completeness with which certain data, both geologic and geographic are presented on the maps; and on the care with which scale, colors, conventions, etc are chosen to give the best results (Eckel, 1902). With the development of technology however, geological maps today are more precise than ever as a combination of accurate satellite imagery, aerial photographs, high tech geological equipment and Geographic Information Systems (GIS) advancements are applied.

2.2.2 Objectives of geological field mapping

Exploitation and planning of an area for certain propose requires the appreciation of basic geology and optimum utilization of a potential area requires that the rocks and soil units are mapped out. The reason why geological field mapping is carried is to create understanding of the spatial distribution and deformation of rock units at the surface and to use as a base map for developing a 3-dimensional understanding of the subsurface geology. In site exploration for the infrastructures of diversion project sites, dam sites of any scale and associated structures sites, the geology of the area has to be thoroughly investigated and understood using a combination of geological method before a decision is made on the selection of that area. Geological mapping is usually the first task in any infrastructure foundations study as the question to what level the site is suitable to be answered using geological studies.

2.2.3 Steps (phases) of geological mapping

Geological mapping requires passing through the following three phases which have a stepwise relationship.

A. Planning step (phase)

This phase is mostly carried out in the office although a short reconnaissance field trip may be included. Once an area has been identified for mapping, it is only rational to learn everything in its regard so as to make a workable field programme. A field programme is a step-by-step guide that outlines the time to be spent in the field and the objectives of each day thus ensuring successful and satisfactory results. Compilation of all available and relevant data is implemented, in an effort to avoid duplication of work and most importantly simplify the field study. The number of days spent during the planning phase may vary based on the purpose of the study, the detail required and extent of the area to be mapped. All possible geological reports and data, including aerial and satellite photographs, maps whether topographical, base maps, legislative boundary maps etc. must be obtained. The most important document however is the official approval by a government office and/or a local community. Without this one may be met with hostility and lack of cooperation by the locals and this may lead to an unsuccessful field study. A budget must also be prepared indicating the number of people to be involved in mapping, their daily allowance, fuel costs if vehicles are to be used, consumables and a miscellaneous kitty should be included. It is wise to involve the local community when employing casuals or language translators as they understand their environment better than anyone else and will thus be quick to pinpoint areas that maybe of interest. Furthermore, they are in the best suited to relate historical events where necessary. Issues that may hinder optimum productivity such as bad weather, wild animals and events of sudden sickness must be taken into consideration and planned for appropriately.

B. Mapping/collecting data

This phase is very important in any geological field mapping and is carried out in the field for the sole purpose of collecting data. This data may be collected in the form of photographs, measurements, notes and physical samples. Therefore, one must be fully equipped with all the necessary tools, be physically and mentally prepared to make note of not only geological features but of the entire surrounding. For example, phreatophytes may be used to locate structures especially when they grow in a certain alignment. When good strategy is involved during the planning phase, unfortunate incidences in the field are addressed promptly with no major disruptions of the field program. If there is possibility of adjusting time schedule, it is advisable to plan for field studies during the dry seasons. Thus it is in the best interest of everyone to commence mapping work very early in the morning so as to accomplish a substantial amount of work before the temperatures rise too high. Thus there should be no strict working hours in the field and this should be agreed upon among the field crew. Teamwork should be encouraged given that more observations are likely to be made and geological contentious issues discussed. A mapping project must be qualitative as well as quantitative. Accuracy whilst taking readings should be emphasized and no amount of information is too much therefore all possible data should be collected.

C. Reporting phase

It is often said that a report is as good as its data and thus the need to collect good accurate data cannot be over emphasized. Ultimately when all possible available data has been collected, it is taken back to the office or laboratory for sorting, interpretation and analysis. This phase is the most challenging of all three as wrong analysis or misinterpretation of data can lead to an inaccurate report and in consequence misinformation. The resulting map or model is in most incidences drawn by persons of another section (for example cartography) who may have had no involvement in the field whatsoever. Thus it is crucial that the data collector, in this case the geologists, work closely with those involved in actual drawing of the map, since they can select the data that is relevant for final presentation in the map and that which is not.

2.2.4 Tools and equipment used in geological field mapping

Geologists of mapping work need a number of items for the field. A hammer is essential and also some chisels. Also essential are a compass, clinometer, pocket steel tape, and a hand lens, notebook, map scales, protractor, pencils and eraser, an acid bottle and a knife. Camera is a must and a small pair of binoculars can be most useful at times, as is a GPS instrument. Sometimes a 30m tape may be needed and a stereo net. If using aerial photographs you will

need a pocket stereoscope; very occasionally a pedometer can be useful, although not essential. You will also need a felt-tipped marker pen and/or timber crayons for labeling specimens.
Finally, you need a rock sack to carry it all, plus a water bottle, emergency rations, a first aid kit, perhaps an extra sweater, your mobile phone, and of course your lunch.

Geologists must also wear appropriate clothing and footwear for the field if they are to work efficiently, often in wet cold weather, when other (perhaps more sensible) people stay indoors; inadequate clothing can put a geologist at risk of hypothermia. A more detailed description of the essential toolss is given below as follows.

***Maps**:* During the planning phase all existing data and maps of the field in question are collected. All suitable maps available whether physical, political, relief, road and topographic should be carried to the field as it is possible that details in one may not be present in another. Most importantly for geological fieldwork, a geological map is expected to be handy especially as a reference. Depending on the areal extent of the field and the detail required, the scale of the map is an important aspect to consider.

***Aerial photographs and stereoscope**:* This 3D imagery tool is very vital for all geological field work especially where large features such as volcanoes, calderas craters etc are involved. In addition photos are also used to pinpoint thermally anomalous areas in geothermal fields when infrared thermography is applied. These photographs are studied with the aid of a stereoscope and are put to use before and during the field study; for planning in the case of the former and for confirmation purposes in case of the latter. Where aerial photographs may not exist, satellite imagery may be used although it may not possess the fine detail as seen on the aerial photographs.

***Compass/clinometer**:* A compass is an instrument used for determining direction and has recently been supplmanted by modern devices such as Global Positioning System (GPS). However, when acquiring a clinometer it is advisable to purchase one that has a built in compass in it. This is for the reason that a GPS can at times malfunction or it may not locate satellites in areas of thick/dense forest cover. A clinometer is an instrument used for measuring inclination, tilt and elevation of rock outcrops. This is particularly important in areas that have been subjected to tectonic movement.

***GPS (Geographic Positioning System**)* is a satellite based navigation system comprising three basic parts; the satellites in space, monitoring stations on earth and the GPS receivers. This equipment is used in geological field mapping for finding ones position, mapping lithologies, tracking structures, measuring elevation, storing sampling points and descriptions of formations when samples are collected. The GPS functions and capabilities are improving rapidly with advancement in technology and as such it is important to purchase one that is relatively modern.

Geological hammer is basic equipment for any geologist as it is the tool used for collecting samples. The best geological hammers are the ones with a piece chisel head made of hardened steel and a rubber coated shock reduction handle. For harder formations, a crack geological hammer forged from fine grain carbon steel with fiberglass shaft fitted with comfortable shock-absorbing rubber grip is recommended.

Hand held lens is used to make the first analysis of rock samples in the field before further analysis is performed in the laboratories. The analysis needs to be detailed and descriptive giving all properties of the sample: rock type, color, texture, identifiable mineralogy, alteration as well as the physical properties such as folding, foliation, intrusions, layering etc.

Sample bags which best suit geological samples are canvas in fabric has sewn in tie tape and a label tag on the outside to insert the sample number and location point. Plastic bags may be used where the sample is soft, disintegrated or wet.

Measuring tape is important for taking actual measurements of lithologies and structures. For instance in lithologies that exhibit layering, it is necessary to measure the thickness of each layer precisely, as this can be used as a correlation tool with other similar sequences that may be encountered.

Field notebook, masking tape, marker pens*:* All important observations must be written down in a concise, orderly and legible manner. The field note book should be hardcover for ease when writing and should easily fit into a pocket. Masking tape and marker pens are used for labeling samples before they are put into the sample bags.

Field camera is the only back up memory in addition to the notebook. It is very important to take photographs of all interesting features ensuring a scale is used in each instance; video clips can also be of use. One must also remember to take a GPS reading for each photograph taken. Photographs are important for descriptive purposes especially during report writing and for making presentations.

Safety clothing**:** These include sturdy shoes, clothes that are tough in fabric preferably jeans or khaki, hat and sunglasses to protect one from the sun a bag pack, plus a raincoat and wellington boots just in case of wet environments. Safety glasses and gloves are important especially when hammering rock samples. Safety clothing should aid one to be more effective by being comfortable and safe at the same time.

First aid kit: Safety in the field is always a priority; nevertheless accidents cannot be ruled out thus the need to be prepared with a first aid kit. Furthermore, it is crucial that at least one person in the field crew be trained in basic first aid techniques.

2.2.5 Methods of geological mapping

Geological mapping is the process of making observations of geology in the field and recording them so that one of the several different types of geological map described produced. The information recorded must be factual, based on objective examination of the rocks and exposures, and made with an open mind: geology is too unpredictable to be approached with preconceived ideas. Obviously the thoroughness with which a region can be studied depends upon the type of mapping on which you are engaged. A reconnaissance map is based on fewer observations than, say, a regional map, but those observations must be just as thorough. Whatever the type of mapping, whatever your prior knowledge of an area, map with equal care and objectivity. For more detailed mapping work guide, use the methods described in below.

I. Traversing

Traversing is basically a method of controlling your progress across an area, so that you do not have to relocate yourself from scratch every time you make an observation at an outcrop.

It is also a method of covering the ground in the detail required by your employer. A traverse is made by walking a more or less predetermined route from one point on the map to another, plotting the geology on the way. Traverses are an excellent way of controlling the density of your observations. They should be planned to cross the general geological grain of the part of the area you are working in, and in reconnaissance work, which is its main use, a number of roughly parallel traverses may be walked across an area at widely spaced intervals. Contacts and other geological features are extrapolated between them. This leads to few complications in area where the rocks are only moderately folded and dip faults are few, but reliability decreases as structures become more complex. Traversing can also be used to map areas in detail where rocks are well exposed, especially those where there is almost total exposure. In such cases, traverses are closely spaced. GPS is an obvious help in traversing.

a. Controlling traverses

Unless traverses are strictly controlled, survey errors accumulate to an unacceptable level. If a traverse made on compass bearings consists of a number of points, either start and finish on known points if possible; otherwise close the traverse by returning to the starting point. Invariably, when you plot this ‘closed’ traverse you will find that the last bearing does not fall exactly where it should do, owing to an accumulation of minor errors of direction and distance measurement. This *closure error* must be corrected by distributing it over the whole traverse, not by fudging the last (point).

b. Cross-section traverses

Whatever mapping method you do use, it can be useful where a succession is doubtful or structurally complex to traverse across the geological grain, plotting a cross-section as you go. Draw it in your notebook or on squared paper, but also show the traverse line on your field map. The advantages of drawing sections in the field are obvious: problems come to light immediately and can be promptly investigated.

c. Stream and ridge traverses

Streams and ridges are features which are usually identifiable on even poor quality maps. Streams often give excellent semi-continuous exposures and in some mountain areas may be so well spaced that a major part of the geology of that area can be mapped by traversing them, especially where slopes are partly covered by colluvium. Position finding on streams is often relatively easy from the shape and direction of bends, and the position of islands, waterfalls and stream junctions, or sometimes by resecting on distant points. In dense mountain rainforest, streams and rivers may be the only places where you can locate yourself, providing of course, the base map itself is accurate, or you are lucky enough to have photographs. Remember GPS is no good in forests.

Ridges, and the spurs which lead off them, may make excellent traverse locations. They can usually be identified easily on a map or aerial photograph. Even in dense forest, ridges may be relatively open, giving opportunity to take bearings to distant points. Exposures are usually good. Most ridges are there because they are the more erosion resistant rock, and in sedimentary rocks tend to follow the strike. Traverses down spurs provide information on the succession although the streams between the spurs may be even better.

d. Road traverses

A rapid reconnaissance of an unmapped area can often be made along tracks and roads and by following paths between them. Roads in mountainous regions, in particular, usually exhibit excellent and sometimes almost continuous exposures in cuttings. In some places, roads zigzag down mountain sides to repeat exposures of several different stratigraphic levels. A rapid traverse of all roads is an excellent way of introducing yourself to any new area you intend to map in detail.

II. Following contacts

A primary objective of mapping geology is to trace contacts between different rock formations, groups and types, and to show on a map where they occur. One way of doing this is to follow a contact on the ground as far as it is possible to do so. In some regions and with some types of geology this is easy; elsewhere it is often impossible because contacts are not continuously exposed. The continuation of contacts beneath drift and other superficial deposits can often be located by plotting structure (or stratum) contours. Sometimes contacts can be followed more easily and more accurately on aerial photographs, using even just a pocket stereoscope than on the ground. The photographs show small changes in topography and in vegetation which cannot be detected on the ground but which indicate the position of the contact even when it is concealed by colluvium or drift. Once traced on the photographs, check the position of the contact in the field at its more accessible points.

Wherever rocks are seen in contact, show the boundary as a continuous line on the map and mark each side of the line with the coloured pencil appropriate to those rocks. Where contacts are inferred, or interpolated by geometric methods, show the boundary with a broken line. Where a contact is concealed, perhaps by scree or alluvium, but is certainly there, show it by a dotted line. However, when tracing a contact, do not forget the ground between it and the next contact, either up the succession or down it; sometimes contacts are close enough to be traced at one and the same time, sometimes they are not.

III. Exposure or line mapping

Mapping by exposures is the main stay of much detailed mapping at scales of 1:10 000 and larger. The extent of each exposure, or group of exposures, is indicated on the field map by colouring them in with the appropriate coloured pencil for that rock type or formation.

Marking the boundaries of very large exposures helps objectivity in the field: outline the exposure, then map within it. If complex, or if there are specifically interesting features to be seen, a large-scale sketch map can be drawn in your notebook. Show groups of exposures which are obviously part of the same outcrop covered thinly by soil as a single exposure. Mark small isolated exposures by a dot with a note or symbol to indicate its nature beside it.

The reason for exposure mapping is that it shows the factual evidence on which your interpretation of the geology will be based; it shows what you have seen, not what you infer. A properly prepared field map should leave no doubt of the quality of the evidence on which it is based. From the foregoing it will be apparent that there is no single mapping method to cover every eventuality. Sometimes you may have to use several different methods in different parts of a large mapping area.

a. Descriptive map symbols

There are some areas where the geology can be mapped only by identifying every exposure in turn; for instance, in Precambrian metamorphic terrains slates pass into phyllites, then to schists, migmatites and gneisses of several different kinds. Many boundaries are gradational and contacts have to be decided by textural and mineralogical characteristics. In these conditions, the usual colour coding used to distinguish formations on your map is inadequate, although it may serve to classify your rocks into broad groups. You must devise a letter code so that you can give a shorthand description of every exposure on your map to show how metamorphic lithologies change and so decide your geological boundaries. You may need to distinguish, say *microcline-porphyroblast coarse grained quartz-albite-microcline-muscovite- biotite gneiss* from other, not quite similar, gneisses. This could be condensed to *M c/gr q-ab-m-mu/bi gn*, where *M* stands for microcline porphyroblast and *m* for microcline in the groundmass, c/gr for coarse grain, etc. Devise your own code and note it in your notebook, so any others working after you can understand your map. Such codes give the *field names*.

b. Form line maps

The subdivision of rocks of an area into mappable units (or formations) may be hampered in some areas by the lack of variety of rock types. Some metamorphic rocks may show outcrop scale size variations in lithology, but may appear monotonous in composition on a larger scale. If formation boundaries cannot be traced across the area, an understanding of the geological structure has to be based on strike and dip readings of compositional layering taken at numerous outcrops. This layering may represent bedding, but could be of tectono-metamorphic origin.

A form line map is an interpretation of the form of the geological structure based on the assumptions of the measured strikes, and dips arise from the sectioning of continuous geological surfaces. Although these maps are somewhat subjective and are drawn freehand, they are useful for the recognition of major changes of strike caused by folding.

IV. Mapping in poorly exposed regions

If an area is poorly exposed, or the rocks are hidden by vegetation, climb to convenient high ground and mark on your map the positions of all the exposures you can see (this is where binoculars prove useful); then visit them. Of all rocks, mica schists probably form the poorest exposures but even they may show traces on footpaths where soil has been worn away by feet, or by rainwash channelled down them. Evidence of unexposed rocks may sometimes be found where trees have been uprooted by storms and in the spoil from holes dug for fence posts or wells, in road and railway cuttings, and from many other man-made, or even animal-made, excavations.

The following are some of the guides which help to identify (estimate) the rocks beneath the forest and soil cover.

- Indications of rocks from soils
- Vegetation guides
- Topography and geomorphology
- Structure (stratum) contours

V. Superficial deposits

Except for valley alluvium, superficial deposits, or drift, are commonly looked upon as a barrier which hides the more interesting solid geology. Unconsolidated superficial deposits are the debris resulting from rock weathering during the formation of the landscape. They include scree (*talus*), which forms unstable slopes of mainly coarse unsorted fragments of broken rock. They also cover 'colluvium', a general term for the rocky hill-side soils shed by rock weathering and the poorly developed soils on the lower slopes. They also include the well-developed and thicker soils on lower-lying ground formed largely by rock weathering in place. Except for scree, these need not be shown on your final map although they appear on your field map as the material which occupies the space between rock outcrops. However, notes can be made, such as ‘red soils’ to justify the concealed continuation of a dolerite dyke; or ‘sandy soils’ to reflect underlying acid rocks.

Alluvium is the transported, washed and sorted result of rock weathering, and it includes everything from boulders and gravel, through sands, to silt and clay. Then there are boulder clays, morraine and fluvio-glacial deposits of many types; beach deposits of many grain sizes; wind-blown sands forming dunes (yes, even in Britain) and fine-grained *loess*. More consolidated, but still superficial weathering products include laterites whilst some spring-fed streams may deposit travertine terraces. All these you do show on your maps.

In general select what is important to the later history of your area and for the principal purpose of the map. If necessary prepare a separate ‘drift map’.

2.2.6 Parameters to consider in geological mapping

2.2.6.1 Geology

When mapping rock types, it is important to have a carefully laid out plan for the entire location so that traverses can be made without leaving out some areas. A careful study of aerial photographs and topographical maps can aid one to avoid areas that are impractical to traverse on foot or impossible to climb because some areas are too steep. In areas where there is thick soil cover, riverbeds, valleys or exposures on hill tops are sort after for outcrops.

Collecting fresh samples is a process performed carefully, ensuring that the sample is not altered. The sample size depends on its intended use, be it for binocular, petrographic, X-ray analysis, whole rock analysis or just for reference. However, a sample should not be too big as to become a problem to carry while in the field. When acquiring a sample for whole rock analysis, it is advisable to break it using another rock from the same formation to ensure that it is not contaminated by metals from the geological hammer.

Geological descriptions must be unambiguous such that other observers given the same sample would describe it the same way (Wyllie, 1999). Moreover, every fine detail must be included whether it is deemed geologically relevant or not.

Strike and dip are a technique of relating the orientation of a plane in three dimensional spaces. The technique is usually applied to the orientation of tilted layers of rock and thus one can easily see the disarticulation a formation has undergone. As mentioned earlier a clinometer is used for these measurements which should be carried out more than once with utmost precision.

I. Rock descriptions

When you have mapped a rock unit for long enough to be familiar with it and its variations, describe it fully and systematically in your notebook. Rock descriptions are essential when you come to write your report later. A rock description made from memory, perhaps weeks later, is unlikely to be accurate or complete. One made in the field describes the rock as seen, with measurements of specific features, and factual comments on those subtle characteristics that are impossible to remember properly later. It also ensures that you record *all* the details needed.

Systematically describe each rock unit shown on your map in turn. Preferably work from the general to the particular. Describe first the appearance of the ground it covers: its topography, its vegetation, land use, and any economic activity associated with it. If the soils are distinctive, describe them too. Next describe the rock exposures themselves: their size frequency and shape; whether they are turtlebacks, pavements, or tors; or rounded or jagged ridges, gentle scarps or cliffs. Comment on joint spacing, bedding and laminations, structures, textures, cleavage and foliation. Support your observations with measurements. Describe the colour of the rock on both weathered and freshly broken surfaces. Weathering often emphasizes textures; note its effect, such as the honeycomb of quartz left on the surface of granites after feldspars have been leached away, which immediately distinguishes silicic from less silicic varieties. Finally describe the features seen in a hand specimen, both with and without a hand lens. Note texture, grain- size and the relationship between grains. Identify the minerals and estimate their relative quantities, bearing in mind the tendency to overestimate the proportion of dark minerals over paler varieties. Name the rock. Where appropriate, prepare a sedimentary section and/or log. A *formation letter(s)* will eventually be assigned to every mapable rock unit, but that is something to be done later. Remember, you can take a specimen home with you, but not an exposure. Ensure that you do have all the information you need before you leave the field.

II. Identifying and Naming Rocks in the Field

A field name should be descriptive. It should say clearly what the rock is, but you cannot name a rock until you have identified it. A field geologist should be able to determine the texture, the relationship between minerals, and estimate their relative abundances in most rocks under a hand lens. It should be able to give some sort of field name to any rock.

A field name should indicate structure, texture, grain-size, colour, mineral content and the general classification the rock falls into, e.g. *thin-bedded fine-grained buff sandstone* and *porphyritic medium-grained red muscovite granite*. These are the full field names but shortened versions, or even initials, can be used on your field map. Avoid at all costs calling your rocks, A, B, C, etc., on the assumption that you can name them properly in the laboratory later: this is the coward's way out. If you are really stuck for a name, and with the finer-grained rocks it does happen, then call it *spotted green rock*, or even *red-spotted green rock* to distinguish it from *white-spotted green rock*, if need be. Ensure, however, that you have a type specimen of every rock named. Sometimes you may find it helps to carry small chips around with you in the field, for comparison.

III. Litho-stratigraphy and Sedimentary Rocks

In the previous section the word formation has been used in a very general sense for a mappable rock unit as a matter of convenience, and for the lack of a better word. However, for formal use there is an accepted conventional litho-stratigraphical hierarchy of terms to describe the grouping of rock units (Holland *et al*. 1978, p. 8). These are described as:

a. Sedimentary formations

A sedimentary *formation* has internal lithological homogeneity, or distinctive lithological features that constitute a form of unity in comparison with adjacent strata. It is the basic local mappable unit. It crops out and can be traced subsurface to other exposures; you show it on your map with a distinctive colour. It is the primary local unit (Holland *et al*. 1978). *For convenience, it may be sub-divided into members*. If a formation has not already been formally named, name it yourself in the approved manner, attaching a place name to the rock name, e.g. *Antalo Limestone Formation*, or for working purposes just call it the *Antalo Limestone*. Avoid terms, such as *White limestone* or *Brachiopod Bed*.

A formation may consist of several *members*, which may not be continuous but have a distinct lithological character. The smallest division of a formation is a *bed*, which is a unit with a well-marked difference from the strata both above and below it. A *group consists* of two or more naturally related formations. A *super-group* consists of two or more associated groups. Groups do not have to be collected into super groups.

b. Stratigraphic sections

Stratigraphic sections show the sequence of rocks in a mapped region, distinguishing and naming the formations and members that comprise them. They show the thickness of the units, the relationships between them, any unconformities or breaks in succession, and the fossils found. It is impossible to find one continuous exposure that will exhibit the complete succession of a region and a complete succession is built up from a number of overlapping partial sections. There may even be gaps where formations are incompletely exposed.

Sections can be measured in a number of different ways and some guide- lines are given here. The first task is to find a suitable place with good exposure. Make measurements of the true thicknesses of the beds, starting at the base of the sequence, and log them in your notebook as a vertical column. In measuring thickness, corrections must be made for the dip of the beds and the slope of the surface on which they crop out. This can be done graphically or trigonometrically. Compton (1966) illustrates several methods of measuring true thickness directly.

Indicate on the stratigraphical section the name and extent of every litho- logical unit, together with the rock types within it. Take specimens of everything logged. Mark and note the names of any fossils found; collect specimen for later identification where necessary. Note the position of every section on your field map. Redraw to scale in camp the sections from your notebook on squared paper. Later the section may be simplified and combined with sections from other parts of your mapping area as a columnar section or a fence diagram. Stratigraphic sections may also include igneous and metamorphic rocks.

c. Grain sizes

Many sedimentary rocks can be classified by their grain size. Anything greater than 2 mm is gravel; anything less than 4 microns is mud; what lies between is sand or silt. Each of these groups is subdivided into coarse, medium and fine, etc. Measure larger grains in the field with a transparent plastic scale placed over a freshly broken surface; use a hand lens with the scale for the finer sizes. Generally, if a piece of rock is gritty between your teeth (no need to bite!), then silt is present, and if grains lodge between your teeth there is fine sand, but that should be visible under your hand lens.

d. Smell

Some sandy rocks also contain clay. Breathe on a fresh surface and note whether it returns a clayey smell. This is not infallible, for if the rock has been too indurated, the clay minerals will have been altered to new minerals. Other rocks, namely those which once had a high organic content, emit a sulphurous smell when hit with a hammer. Iron staining indicates iron cement.

e. Hardness

Always test a very fine-grained or apparently grain less rock by scraping the point of your hammer across it. If it scratches, it is probably a sedimentary rock, if not it may be a chert or hornfels, or an igneous or pyroclastic rock. Some white, cream or grey rocks can be scratched with your fingernail. They are probably gypsum or anhydrite; possibly even rock salt, but one lick can settle that!

f. Acid

Every geologist should carry a small bottle of 10% hydrochloric acid in the field. To use it, break off a fresh piece of rock, *blow off any rock dust*, and add *one* drop of acid. If the reaction is vigorous, the rock is *limestone*. If it does not fizz, scrape up a small heap of rock powder with your knife and add another drop of acid to it. Gentle reaction indicates *dolomite*. Many carbonate rocks contain both calcite and dolomite, so collect specimens for staining when you return to base. Remember, however, that some rarer carbonates react to acid too. Do note that one drop of acid is enough to test for reactions. Do not flood the surface with it, all you need is a very small plastic bottle, the type used for eye-drops.

g. Fossils

Fossils cannot be considered in isolation from their environment. All the features found in a fossiliferous rock must be recorded if you are to gain the full benefit from the fossil itself. Note their abundance in each fossiliferous horizon of the locality. Once you have discovered a sequence containing specific fossils in one part of your mapping area, you may then find that you can use it for mapping on a wider basis, especially where you have a series of repeated sequences or cyclothems. The fossils will tell you which part of the series you are in. Again, where you have beds of great thickness, your fossiliferous horizon will tell you where in that bed you are. It can even tell you the displacement, where both sides of a fault are in the same rock type. Mark rich, or important, fossil localities on your map with a symbol, so that others can find them again later.

IV. Igneous Rocks

Phaneritic igneous rocks are easily recognized and the acid to intermediate (leucocratic) varieties can usually be readily named. Dark-coloured (melano- cratic) phanerites are perhaps a little more difficult to identify, but you can usually put some field name to them which is nearly correct. Before you go into the field, try to look at specimens of the types of rock you expect to encounter; if possible, those from the area you are going to.

Grain-size in phaneritic rocks: Grain-size terminology in igneous rocks differs from that used for sediments, its grain size class and range is shown on the table 2.1 below:

Table 2-1: Grain size description terminology of igneous rock

Description	Size of grains in mm
Coarse–grained	> 5
Medium–grained	1 – 5
Fine-grained	< 1

Use the terms coarse, medium and fine when discussing a rock, but in formal descriptions state grain sizes in millimeters. If a rock is porphyritic or porphyroblastic, remember to quote the size of the phenocrysts or porphyroblasts too; a phenocryst or porphyroblast 10 mm long may appear to be ‘large’ in a fine-grained rock, but not in a coarse one.

Igneous mineralogy

When naming a rock, identify the principal minerals and estimate their relative abundance.

a. Aphanitic Igneous Rocks

Aphanitic igneous rocks can be difficult to name in the field. Hard and compact, at first sight they appear to give little indication of their identity. Divide them into light-coloured aphanites, ranging up to medium red, brown, green and purple; and darker aphanites covering colours up to black. Use the old term felsite for the first group and mafite for the second. Table 2.2 shows how the two groups divide. Careful examination of aphanites under a handlens usually gives some pointers to their identity, and many contain phenocrysts, which also helps. Basalt is by far the commonest of all black aphanites. In the field, refer to the 'spotted black rock' type of terminology if all else fails.

Table 2-2 group of aphanitic ignous rocks

Felsites	Mafites
Rhyolites	Andesites (a few)
Dacites	Basalts
Trachytes	Picrites
Andesites (most)	Tephrites
Phonolites	Basanites
Latites (trachyandesites	

b. Veins and Pegmatites

Quartz veins are common and should give no trouble in identification. Some are deposited by hydrothermal solutions along fractures and may show coarsely zoned structures and, sometimes, crystal lined vughs. Others have been formed by replacement of rock and may even show 'ghosts' of the replaced rock with structures still parallel to those in the walls. Some veins are clearly emplaced on faults and may enclose breccia fragments; some may contain barite and fluorspar and even sulphides. If pyrite is present, check for ore mineral. However, not all veins are quartz veins. Some contain calcite, dolomite, ankerite or siderite, or mixtures of them, and they may be mineralised too.

Pegmatites always have igneous associations. They are usually, but not exclusively, of granitoid composition. Grain size may be from 10 mm upwards to over metre size. 'Granite pegmatites' fall into two main groups: *simple* and *complex*. Simple pegmatites are usually vein-like bodies consisting of coarse-textured quartz, microcline, albite, muscovite, sometimes biotite and, rarely, hornblende. Complex pegmatites can be huge, some may be tens of metres long, and often podlike in shape with several distinct zones of different composition around a core of massive quartz. They may be mineralised, with beryl, spodumene, large crystals of commercial muscovite, and various other micas, including zinnwaldite and lepidolite.

c. Igneous Rocks in general

Always examine an igneous contact thoroughly. Look at both sides carefully and make sure that it is not an unconformity or a fault contact. Note any alteration and *measure* its extent: a 'narrow' chill zone will mean little to the reader of your report. Sketch contact zones and sample them. Contact metamorphism converts mudstones to hornfelses, hard, dense fine-grained rocks, often spotted with aluminium silicates. They can be difficult to identify; map them as appropriate, e.g. *grey hornfels*, or *spotted black hornfels*, or *garnetiferous green hornfels*. Sandstones are metamorphosed to quartzite near contacts. Carbonate rocks become *tactites or skarns*, diverse mixtures of silicate minerals. Search skarns with special care for ore minerals, for they are very susceptible to mineralisation. Examine contacts between lavas (and the rocks both above and below them) closely and do not forget the contacts between individual flows.

Few large intrusions are homogeneous, yet many maps give that impression, for they are often shown on a map by only one pattern or colour. Map the interior zones of an intrusion with the same care you would give to an equivalent area of sedimentary rocks. Boundaries between phases may be irregular and gradational, but differences in mineral composition and texture, and very often flow-banding, can usually be seen if looked for. Map them. Map also all dykes and veins in intrusions, or at least note their frequency and strikes if small. Record all joint patterns.

d. Pyroclastic Rocks

Treat pyroclastic rocks as if they were sedimentary rocks and apply the same rules when mapping them. They are important markers in sedimentary sequences because they may be deposited over wide areas in relatively short periods of time. Pyroclastic materials are essentially glassy ashes. If unconsolidated they are called *tephra*, when consolidated *tuff*. *Agglomerates* are pyroclastics composed mostly of fragments larger than 64 mm, *lapilli tuff* of fragments (usually rounded) of 64 mm down to 2 mm, and *ashy tuff* anything below 2 mm. *Welded tuffs* are those in which the ashy fragments fused during deposition. *Ignimbrite* is a special name reserved *only for* rhyolitic welded tuffs. Name tuffs, where possible, for their related lavas, e.g. *andesite tuff*, or *ashy andesite tuff*, but many fine-grained varieties are difficult to identify in the field and more non-committal names are justifiable. Some tuffs are so glassy, or even apparently flow-banded, that they can be mistaken for lavas in the field. Tuffs tend to devitrify to give spherulitic and perlitic textures. Many weather easily to industrially useful products, such as *bentonite* and *perlite*.

V. Metamorphic Rocks

Contact metamorphism has been dealt with under igneous rocks. Here we are concerned only with rocks resulting from *regional* metamorphism. Two factors need to be considered when mapping them: the original lithology/ stratigraphy, and present lithology. Whenever possible, map them separately.

Sedimentary rocks change with increasing metamorphism, first to slates, then to phyllites, schists and gneisses. Igneous rocks deform and recrystallise to genisses or schists and many basic igneous rocks, including volcanics, become *amphibolites*.

Name *slates* for their colour, such as brown, green, grey, blue or purple and for their recognizable minerals, e.g. *pyritic black slate* or *green chiastolite slate*, and do remember that most slates are not hard roofing-quality slates. *Phyllites* cleave more readily than slates, leaving lustrous faces shining with sericite scales.

Geologists seldom agree where to put their boundary between phyllites and schists in the field: the division tends to be subjective. In general, if *individual* mica or chlorite flakes can be clearly seen, call it a schist, if not, it is a phyllite. *Mica schist* is a common ‘sack name’. Where possible, define ‘mica schists’ as *chlorite schist, muscovite schist, biotite-garnet schist*, etc., but not all schists are micaceous; there are *actinolite schists, tremolite schists*, and many others. Unfortunately, schists tend to weather easily and so are often poorly exposed.

a. Naming metamorphic Rocks

Gneisses are medium to coarse-grained foliated rocks in which bands and lenses of different composition alternate. Some gneisses split roughly parallel to their foliation owing to the alignment of platy minerals, such as micas; others do not. Always qualify the word *gneiss* by a compositional name when first used: not all gneisses are granite gneisses as is too often assumed. Like all other rocks, a locality name can be used as a prefix, or even a more general name, such as *Lewisian gneiss*, to denote gneisses of a certain age.

Gneisses may also be named for their textures, such as *banded gneiss*. Some may contain apparent phenocrysts or *augen*. They may be cataclased *augen*, or they may be *porphyroblasts* of large new crystals growing in the rock, perhaps replacing former augen. You probably cannot tell which until seen in thin section, but *augen gneiss* is a convenient field name in either case, even if not always strictly correct.

Migmatites are, literally, mixed rocks. They contain mixtures of schistose, gneissose and igneous-looking material. Treat them in the same way as other gneisses: name them for composition, texture and structure. For all these rocks, measure the dips and strikes of foliation and the direction and amount of plunge of any minor folds.

b. Contacts

Contacts between many metamorphic rocks are just as sharp as those between most sedimentary rocks or igneous rocks. Some, however, may be gradational, especially within schists and gneisses. Identify every exposure compositionally when mapping them so that gradational boundaries can be inferred where necessary.

c. Foliation

Where structure is fairly regular, map the cleavage, schistosity and other foliations at much the same density as for sedimentary rocks. If structure becomes so complex that it is impossible to show it adequately on your map, map it at a larger scale, or make numerous sketch maps and notebook diagrams.

In addition to foliation, there are many other structures which need to be mapped in metamorphic rocks. These include the plunge and trend of any minor folds, whether in bedding, cleavage or other foliations, or even in pigmatic veins.

2.2.6.2 Geological landforms

Geological landforms refer to features such as mountains/hills, calderas craters, canyons etc. The magnitude of these formations (usually large) can be estimated from aerial photographs and maps prior to the actual field work. The dimensions of these formations must therefore be obtained by actual measurement. The recommended method is to traverse the formation on foot while using a GPS to track the movements. For example, to map the dimensions of a caldera it is in best practice to first measure the rim by walking around it ensuring the GPS is in tracking mode. Since the elevation will be known by walking up the caldera one should then walk into it for further measurements. All lithologies should be mapped both in the interior and exterior of such formations. In some instances landforms such as calderas and craters may not be obvious on the surface due to erosion and weathering which may leave only a few exposures on the surface. If such outcrops are ignored it would be difficult to realize the connection if another exposure of the same kind is found in another location. Needless to say, a structure can easily be left out if work is carried out hurriedly.

2.2.6.3 Geological structures (Faults and fractures)

Faults and fractures always indicate movement within the earth's crust. They are distinguished from each other by whether displacement (lateral or vertical) has taken place; faults show displacement whilst fractures do not. Differences aside, the mapping of these structures needs to be done meticulously due to the fact that these structures may easily be confused with simple erosional surfaces. Erosional surfaces may sometimes be an indication of an underlying structure. The disparity is whether any observable disturbance can be seen in the bedrock. Structures like geological features may not always be obvious on the surface. In volcanic areas, they may manifest as volcanic centers or fumaroles oriented in a linear direction which become obvious once a map is compiled. Thus it is imperative to be accurate in measuring all formations and structures with well calibrated equipment so that if extrapolations are to be made for modeling purposes the error margin is very slim.

2.2.6.4 Cross-Sections

No geological map can be considered complete until at least one cross-section has been drawn to show the geology at depth. Cross-sections explain the structure of a region far more clearly than a planimetric map. They may be drawn as adjuncts to your fair copy map and simplified as text illustrations in your report. In addition to cross-sections, columnar sections can be drawn to show changes in stratigraphy from place to place, or ‘fence’ or ‘panel’ diagrams to show their variations in three dimensions. Refinements in three-dimensional illustrations include block diagrams, which show the structure of the top and two sides of a solid block of ground, and models to aid interpretation, such as ‘egg-crates’.

2.2.7 Compiling geological maps

With the aid of GIS (Geographic Information Systems), maps of good quality and great accuracy are produced. Evidently it is important to consult with the GIS department before field work to calibrate the GPS in format/units that can easily be used with their software. Nonetheless, it is important for a geologist to draw a simple sketch map during field work for comparison purposes.

2.2.8 Interpreting geological maps

Once a map is complete, the most important task ahead is interpreting it correctly. A map is basically a visual summary of an entire report and the two should complement each other. When the interpretation process is at hand, it is crucial to have group discussions with all those involved in the actual fieldwork. It is also deemed important to have discussions with those who are experienced in field of geology as well as other related scientific fields.

2.3 GEOTECHNICAL INVESTIGATION

2.3.1 General

Prescribing the necessary site investigations in general terms is hardly possible; because the selection of appropriate tests depends to a large extend to the general knowledge of the subsoil conditions at the project site and the importance and special requirements of the structures to be raised.

For important structural projects programming and execution of site surveys is generally a growing process, by which site investigations develop from reconnaissance surveys to detailed fact-finding investigations (parameters of soil and rock properties). However, to arrive at an economic and effective site investigation program, the geotechnical engineer/foundation engineer should first collect all relevant data from the client and other sources, which is the most difficult part of foundation engineering. Much depends on the skill, feeling and experience of the foundation/geotechnical engineer, because the foundation design starts in fact at the very moment of the first discussions with the client, who is in most cases not familiar with soil/rock mechanics and the foundation/geotechnical engineer not (yet) familiar with the project.

The foundation/geotechnical engineer should lead the discussions in order to extract from the client all relevant design criteria and assess the importance and relevancy of each criterion.

Below are some aspects will be discussed, however these should not be regarded as a complete check-list, or a binding schedule to arrive at the necessary considerations for a good investigation program. These aspects are based on the three main subjects mentioned in paragraph.

- Type of the project (structures);
- Conditions of the site and its environment;
- Boundary conditions and economical limitations.

In our case, the type of infrastructure associated with small scale Irrigation projects,Diversion structure or headwork, pump station, Canals, Ponds for night storage, Crossing structure

2.3.2 Objective of the investigation

Objectives of the engineering geological and geotechnical investigation process are:

- Identifying and characterizing soils and rocks of the project site (surface & subsurface)
- Extension and thickness of the soil and rock strata within the influence zone of the proposed project structure's foundation.
- Whether the soils and rocks under a project structure site can safely support the proposed project structures
- Groundwater conditions, considering the seasonal changes and the effects of extraction due to construction or development.
- Physical and engineering properties of the soil and rock formations, and effect of groundwater chemical contents on the coming structure.
- Impact of any planned excavation, grading or filling
- Hazardous conditions, including unstable slopes, active or potentially active faults, regional seismicity, floodplains, ground subsidence, collapse, and heave potential.
- Ground response to changing natural conditions and construction or development brought about by surface loadings from structures, un-loadings by surface or subsurface excavations, or un-loadings from the extraction of mineral resources.
- Availability of suitable construction materials for construction of the project within a reasonable distance in sufficient quantity.

Planning the investigation of diversin structures site /headwork site

Geological / geotechnical investigation activities vary from project to project based on type, components, phase and size of the project, and also the geology of the site. Consequently, planning the investigation through considering the necessary project and site conditions will help to obtain necessary result efficiently with in a proper time. The following basic conditions have to be considered planning geotechnical investigation and tastings for projects. To get in to planning site investigation, information on the proposed project development should be gathered. Preliminary investigation should be carried out involving desk study, site inspection, image interpretation and reconnaissance site visit. The information which guides on methods of investigation planning include such as:

- Study phase of the project
- Project location and included components and dimension of the proposed project
- Design parameters of the projects required to be fixed.
- Previous works and Existing documents, if any

Size of the project is an important input for planning the investigation program in order to set down precisely what the investigation should achieve for that particular. In our case, Diversion structures **and simple intakes, crossing and canal and other small Distribution of hydraulic structures associated with Small Scale Irrigation Projects (SSIPs).** The project specific geotechnical investigation objectives shall be defined in such a way that they can meet projects geotechnical parameters to be verified. Check lists of parameters to be verified for Diversion project structural components are presented below.

a. Check list for Diverssion site, simple intakes and associated structures' foundations :

- Location to suit topographic and geologic conditions
- Good understanding of flow under the floor of structure or seepage (failure due uplift and piping) i.e. safe exit gradient.
- Knowledge of permeable and impermeable foundation condition; and foundation stability and suitable foundation material (understanding of foundation design).
- Bearing capacity, settlement problem or compressibility of foundation material
- Bank stability and erodibility: Scouring Potential of the river bank at the abutments (Steeling basin erodability)
- Excavation depth, possible method of excavations
- Lateral pressure condition: Stability of excavation, support, & dewatering requirements.
- Location of other alternative diversion structures sites, if there exist
- River valley, geological x-section along the head work

b. Check list for canals, ponds and other conveyance structures foundations

- Canal alignment engineering geological conditions
- Seepage loss and need for lining/ or other improvement
- Associated failure risks such as: scouring of the canal bank, Uplift of the structure, piping failure, canal slope failure and collapse of the canal material
- Shear failure of the ground
- Excavation condition of the canal route and method of excavation of different canal route sections
- Economical use of locally available construction materials (excavated materials)
- Excavation method(s) and possible use for excavated materials.
- Stability of excavations, need for temporary/permanent support.
- Longitudinal geological profile with test points
- Workability

c. Check list for crossing structures foundations (cross drainage structures)

- Bearing capacity
- Scouring potential
- Foundation stability
- Suitability of groundwater for concrete
- Geological profile with test points
- Workability

d. Check list for availability of construction materials

- Earth fill, for replacing unsuitable canal and pond embankments (for canal fill, lining)
- Rockfill for bank erosion protections
- Rock for massonary
- Crushed rock aggregates material
- Sand and water sources
- Quality, quantity, proximity and location, ownerships, environmental, accessibility

Methods of geotechnical investigations

The method of investigation depend on complexity of the area, objective of the investigation, size of the project, unfavorable geologic conditions, available previous related data and the check lists of the parameter required for safe designing of the proposed structure on the site. After identifying all the necessary list of issues in line with the project under considerations, one could have clear views to plan methodologies and procedures to be included in a site investigation program, in order to address the issues which are presented in the check lists. Moreover, the sequence of investigation methods will also have an effect for improving the volume and quality next investigations method. Accordingly, the most accepted sequence for the investigation method for diversion project site investigation starts with Geological mapping then, excavation of test pits, trenches and augers. In case when geophysical survey and exploratory borehole drilling are required following the complexity of the site; geophysical survey has to be conducted before investigation through test pit excavation and then exploratory drilling will be the final method of investigation.

Issues of the site investigations to be addressed through engineering geological mapping are:

- Description of soil and rocks
- Identifying and characterizing geological structures
- Identifying and classifying land slide porn areas
- Classifying the rocks and soil units in to different geotechnical units based on the engineering properties weathering grades of rocks.
- Collecting samples for laboratory tests
- etc

Guide lines on detail procedures and steps of site investigation during geological mapping are presented on the section 2.2 above.

2.3.3 Test pit excavation

Excavation of test pit is used on the ground type that can temporarily stand unsupported and in suitable conditions. For practical reasons, the maximum depth of excavation by hand is up to 5 m. Minimum top width size of test pits is 1m y 1.5m to the maximum is being 1.5m by 2m. Where personnel are required to enter pits, it is essential that the sides are safe or made safe, particularly from sudden collapse, by supporting the sides. Shovel, crowbar, bucket, pick, ladder, rope, pulley and dewatering systems are tools required for excavating test pits and trenches manually.

Ideally, the support system should consist of purpose-made metal frames that can be quickly inserted and extracted. Alternatively, it may be possible to excavate the sides to a safe profile by means of a series of benches.

By providing access for taking undisturbed and disturbed samples and carrying out in-situ tests, shallow trial pits permit the in-situ condition of the ground to be examined in detail both laterally and vertically; they also provide a means of determining the orientation of discontinuities in the ground. The field record should include a plan giving the location and orientation of the pit with details of which face(s) was logged, and a dimensioned section of each side and the floor. Whenever possible, the record should include photographs.

Shallow trial pits can readily be extended into trenches in order to trace any particular feature, and in suitable ground this method is very efficient and economical. Shallow pits without side support can be used for making a rapid check on the condition of the ground. It may be unsafe for personnel to enter a pit but, working from ground surface, the technician can prepare a visual log of the strata and take disturbed samples using the excavator bucket. Tube samplers can be driven into the floor of the pit, using a jarring link and drill rods, and then extracted by the excavator. In-situ testing, such as the vane shear strength test, Dynamic cone penetration tests (DCP), permeability tests can also be carried out to establish the bearing capacity and permeability conditions of the soil strata in the pits.

Pits that are unsupported may collapse soon after being dug, so any logging, sampling and in-situ testing should be carried out immediately after the pit has been dug. It is advisable to backfill pits as soon as possible after logging, sampling and testing have been completed, since open pits can be a hazard to the general public. There can be advantages though in leaving pits fenced, shored and open (at least overnight and possibly up to 7 days), as this can allow the excavated surface to partially dry, exposing fissures and fabric better than immediately after excavation

Figure 2-1: Excavated test pit on logging and describing

Deep trial pits and shafts are normally constructed by hand excavation using traditional methods for supporting the sides. In suitable ground conditions, shafts approximately one meter in diameter can be bored using large power-driven augers. Temporary liners are used to support the sides in unstable ground, and to provide the necessary protection for personnel working in the shafts. When making inspections, however, it is necessary to expose as much of the ground as possible, and considerable judgment and experience is required, since the excessive use of liners can lead to delay and to additional danger as a result of the build-up of water pressure behind the liners. Suitable ground conditions are those that can be bored with a minimum use of temporary liners and in these conditions augered shafts provide a fast and economical expedient for inspection, sampling and testing in-situ. For many types of ground excavations below the water table present serious problems both in maintaining a dry investigation and in stabilizing the sides. For such circumstances, the water table may be the maximum depth for which this method is feasible.

The hand auger boring method uses light hand-operated equipment. The auger and drill rods are usually lifted out of the borehole without the aid of a tripod, and no borehole casing is used. Boreholes up to 200 mm diameter may be made in suitable ground conditions to a depth of about 5 m. The method can be used in self-supporting strata without hard obstructions or gravel-sized particles. Hand auger boreholes can be used to obtain disturbed samples, small open-tube samples and for groundwater observations.

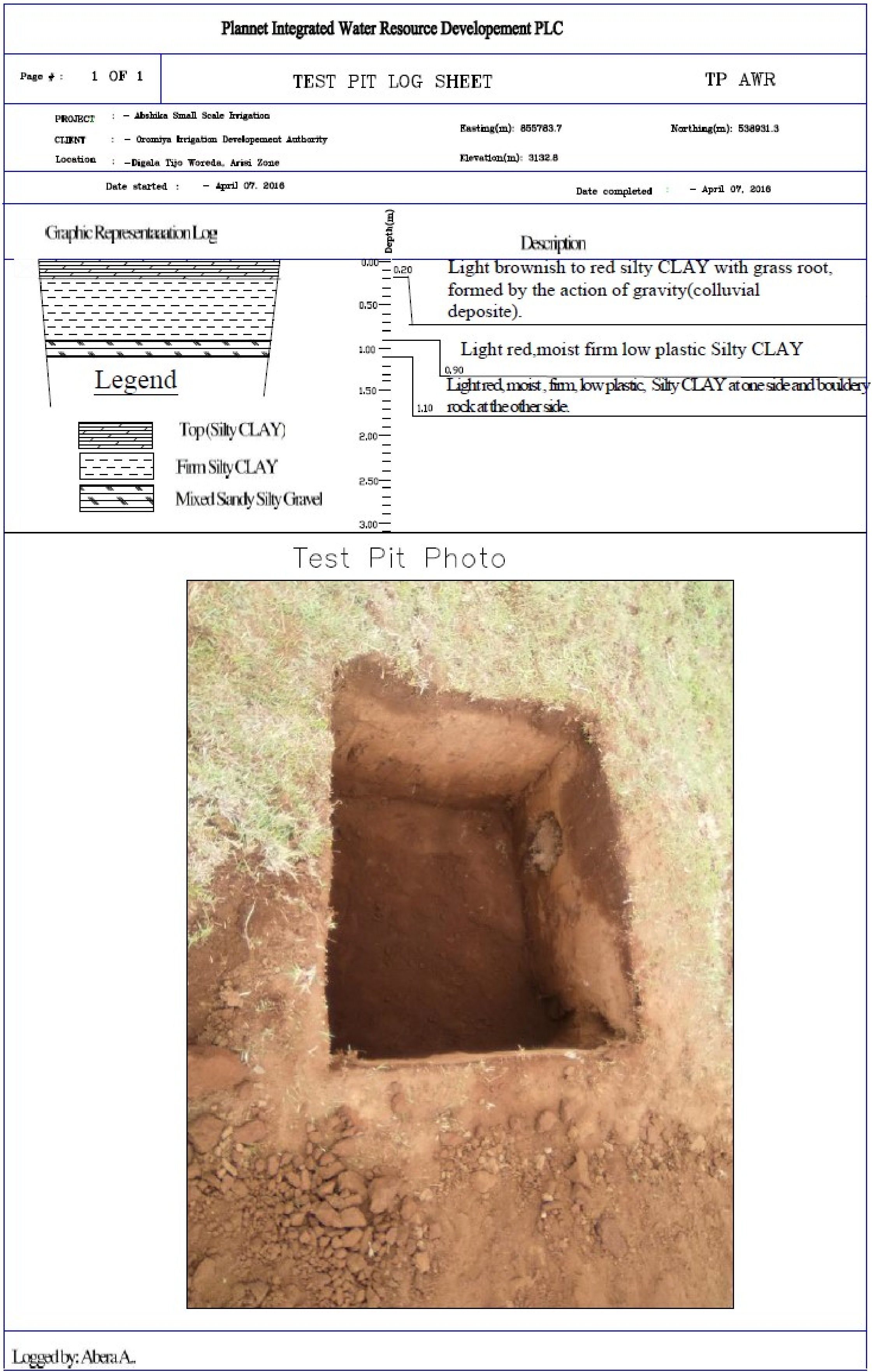


Figure 2-2: Typical test pit (trench) log format used for one of SSID project

2.3.3.1 Describing and logging of soil

Soil description/identification is the systematic, precise, and complete naming of individual soils in both written and spoken forms (ASTMD-2488,AASHTOM145), while soil classification is the grouping of the soil with similar engineering properties into a category based on index test results; e.g., group name and symbol (ASTMD-2487,AASHTOM145). It is important to distinguish between visual identification and classification to minimize conflicts between general visual evaluations of soil samples in the field verses a more precise laboratory evaluation supported by index tests. Group symbols associated with classification should not be used in the field. A visual description in the field is often subjected to outdoor elements, which may influence results. Classification tests can be performed by the laboratory on representative samples to verify identification and assign appropriate group symbols. If possible, the moisture content of every sample should be performed.

The soil description should include the following as a minimum:
Consistency (for fine-grained soils) or density adjective (for coarse-grained soils)
The consistency of fine-grained soils and apparent density of coarse-grained soils are estimated from the blow count (*N*-value) obtained from Standard Penetration Tests (AASHTOT-206, ASTMD1586). The consistency of clay sand silts varies from soft to firm to stiff to hard. The apparent density of coarse-grained soil ranges from very loose to firm to dense and very dense Suggested guidelines in table 2.20 are given for estimating the in-place consistency or apparent density of soils from N-values.

Table 2-3 : Evaluation of consistency of fine and coarse grained soil based on SPT N (can be used converted DCP data) value

Fine grained soil		Coarse grained soil	
N – value	Consistency	N – value	Apparent density
0 – 2	Very soft	0 - 4	Very Loose
2 – 4	Soft	4 – 10	Loose
4 – 8	Firm	10 – 30	Firm
8 – 15	Stiff	30 – 50	Dense
15 – 30	Very stiff	> 50	Very dense
> 30	Hard		

Judgment remains an important part of the visual identification process. Mechanical tools such as the pocket (hand) penetrometer, and field index tests (smear test, dried strength test, thread test) are suggested as aids in estimating the consistency of fine grained soils.

- Water content condition adjective

The amount of water present in the soil sample or its water content adjective should be described as dry, moist, or wet as indicated in Table 2-21.

Table 2-4: Terms to be used for description of moisture contents of soil

Description	Conditions
Dry	No sign of water and soil dry to touch
Moist	Signs of water and soils relatively dry to touch
Wet	Signs of water and soil wet to touch; granular soil exhibits some free water when dandinfied

Color description

The color should be described when the sample is first retrieved at the soil's as-sampled water content (the color will change with water content). Primary colors should be used (brown, gray, black, green, white, yellow, and red). Soils with different shades or tints of basic colors are described by using two basic colors; e.g., gray-green. When the soils marked with spots of color, the term “mottled” can be applied. Soils with a homogeneous texture but having color patterns which change and are not considered mottled can be described as “streaked”.

Minor soil type named with

"y" is added if fine-grained minor component is less than 30 percent but greater than 12 percent or coarse-grained minor component is 30 percent or more.

- Descriptive adjective for main soil
- Particle-size distribution adjective for gravel and sand
- Plasticity adjective and soil texture (silty or clayey) for inorganic and organic silts or clays
- Main soil type name (all capital letters)
- Descriptive adjective “with” for the fine-grained minor soil type if 5 to 12 percent or for the coarse- grained minor soil type if less than 30 percent but 15 percent or more (note some practices use the descriptive adjectives “some” and “trace” for minor components).
- Descriptive term for minor type(s) of soil
- Inclusions (e.g., concretions, cementation)
- Geological name (e.g., Holocene, Eocene, Pleistocene, Cretaceous), if known, (in parenthesis or in notes column

The various elements of the soil description should generally be stated in the order given above. For **example:**

Fine-grained soils: Soft, wet, gray, high plasticity CLAY, with f. Sand; (Alluvium)

Coarse-grained soils: Dense, moist, brown, silty m-f SAND, with f. Gravel to c. Sand; (Alluvium) When changes occur within the same soil layer, such as change in apparent density, the log should indicate a description of the change, such as “same, except very dense”.

2.3.3.2 In situ strength tests the pits

I. Dynamic Cone Penetration Test (DCP)

Dynamic cone penetration test is an important test, which is normally used to determine the relative resistance offered by the different soil layers. The tests can be conducted on the ground surface as well as in the test pits depending on the variation of the soil formations, see Figure 2.3 below. The cone is fixed to the bottom of a rod by pushed fit. The cone is driven into the ground in the same way as a SPT is performed. The maximum depth suggested for this test is about 6 m.

Familiar way of doing analysis of DCP test results is by just by plotting all the DCP test result recorded on a standard field data sheet with the depth of penetration against number of blows using a spreadsheet. The slopes of the curves represent the penetration depth per number of blows (Fig. 2.5).

The rate of change of the slope of the curves or the penetration depth per number of blows revealed the different soil layers and their relative density or consistency. The DCP values obtained for different soil types then converted to SPT N-values/300mm following the correlation developed by Transport Road Research Laboratory (TRRL), UK, Overseas road Note (ORN) 9, Design of small bridges (Table 2.23), to compute the bearing capacity using Meyerhof's equation (cited in Bowels, 1988).

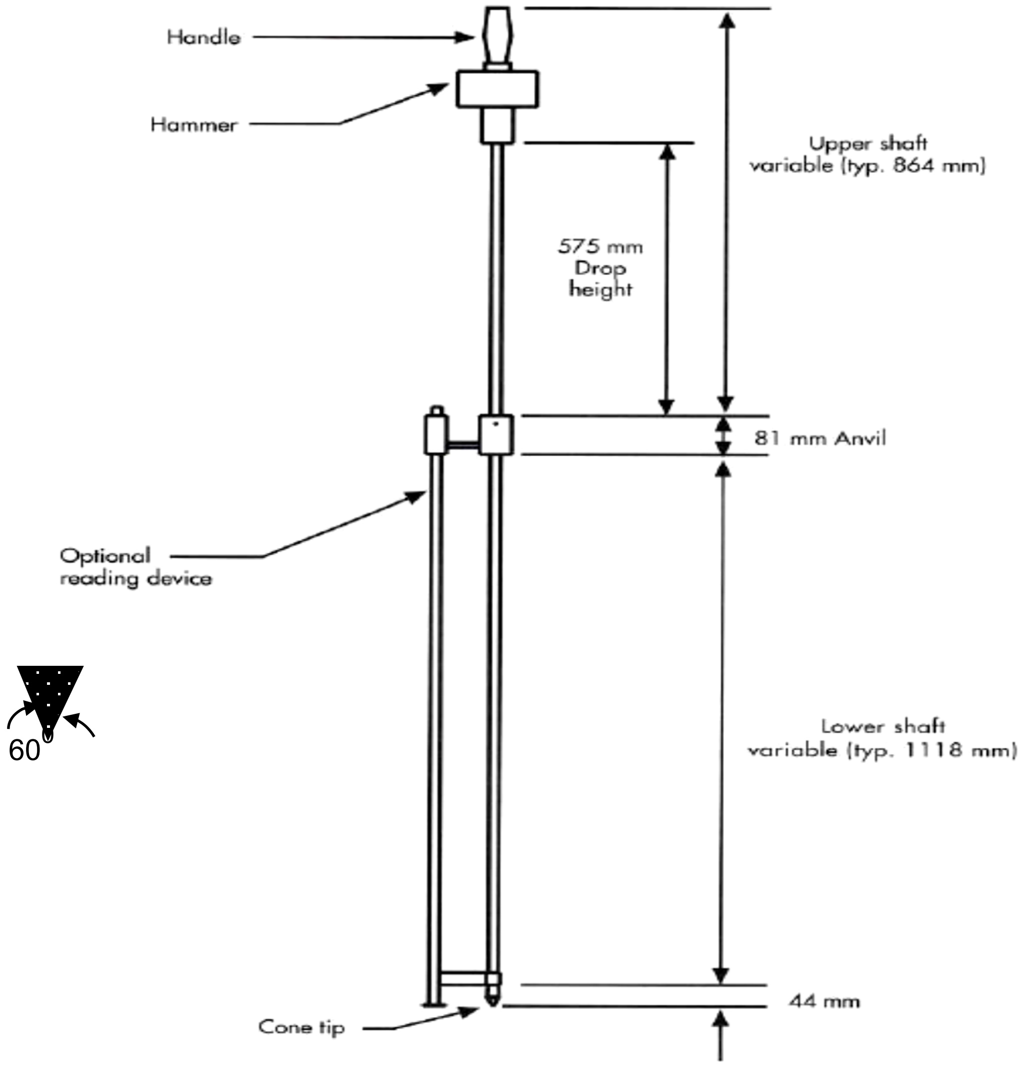


Figure 2-3: Dynamic cone penetration testing instrument, cone and extensions.

Figure 2-4: Assembling the dynamic cone penetrometer inside test pit to conduct the test, TR-39

Table 2-5: Typical format and example for DCP data collection and analysis

Project:____________________
Section/Test pit: ___________________
Test started at (starting layer) :_
Starting depth from the ground:_

Date :____________________
Northing: __________________
Easting:____________________
Elevation:___________________

No. of blows	adjusted depth, mm	Meter reading, mm	Reference	Increment, mm/blow	CBR
0	0	210	210		
1	52	262		63	2
2	115	325		65	2
3	180	390		62	2
4	242	452		44	3
5	286	496		40	4
6	326	536		44	3
7	370	580		50	3
8	420	630		38	4
9	458	668		30	6
10	488	1260	1290	30	6
11	518	1230		40	4
12	558	1190		40	4
13	598	1150		30	6
14	628	1120		30	6

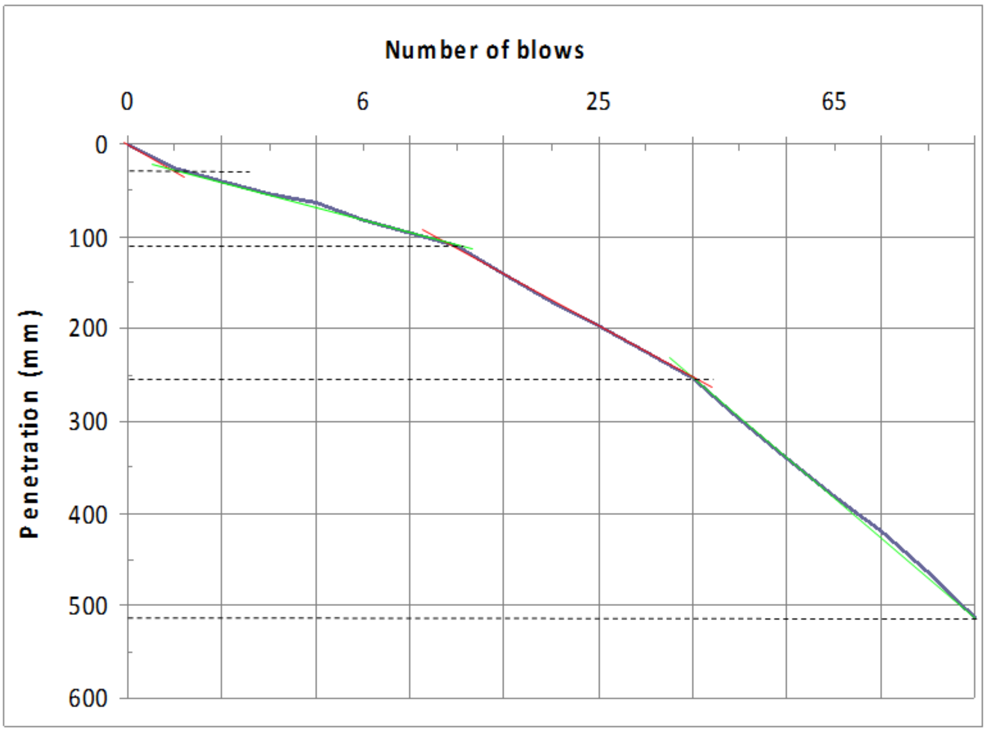


Figure 2-5: Dynamic cone penetration testing instrument, cone and extensions.

Table 2-6: Typical correlation between DCP and SPT values after,TRL, ORN 9, Design of small bridges.

DCP value mm/blow	SPT N value blows/300mm
5	50
6	44
7	38
8	33
9	28
10	24
12	22
14	18
16	16
18	15
20	14

II. Van shear tests

This test is useful in determining the in-place shear strength of very soft and sensitive clays, which lose a large part of their strength when even slightly disturbed by the sampling operation. The strength parameter obtained is consolidated- un drained shear strength, Cu.

In most cases a hole is drilled to the desired depth, where the vane shear test is planned to be performed and the vane is carefully pushed into the soil. A torque necessary to shear the cylinder of soil defined by the blades of the vane is applied by rotating the arm of the apparatus with a constant speed of 0.5 degree/sec. The maximum torque is then measured from which the shearing strength is determined.

From the measured maximum torque one may estimate the shearing resistance of the tested clay from the following formula

$$\tau = \frac{T}{\pi\left[D^2\frac{H}{2}+\frac{D^3}{6}\right]} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.1)$$

where T = Torque
D = Diameter of Vane
H = Height

Since for quick condition $\tau = C_u$, one ultimately arrived the in-situ value of cohesion

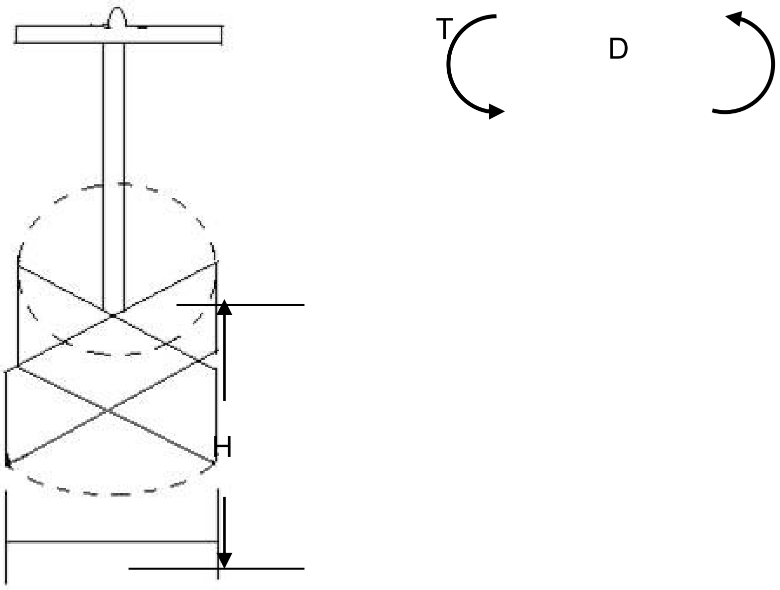


Figure 2-6: Vane shear test equipment

2.3.3.3 Insitu Field Measurement of Permeability

The permeability (k) is a measure of how easily water and other fluids are transmitted through the geo-material and thus represents a flow property. In addition to groundwater related issues, it is of particular concern in geo-environmental problems. The parameter k is closely related to the coefficient of consolidation (cv) since time rate of settlement is controlled by the permeability. In geotechnical engineering, we designate small k = coefficient of permeability or hydraulic conductivity (units of cm/sec), which follows Darcy's law:

q = k*i*A ………………………………………………………………….. (2-2)
Where q = flow (cm3/sec),
i = dh/dx = hydraulic gradient, and
A = cross-sectional area of flow.

Laboratory permeability tests may be conducted on undisturbed samples of natural soils or rocks, or on reconstituted specimens of soil that will be used as controlled fill in embankments and earthen dams. Field permeability tests may be conducted on natural soils by a number of methods, including simple falling head, pumping (drawdown), slug tests (dynamic impulse), and dissipation tests. A brief listing of the field permeability methods is given in Table 6-11.

Table 2-7: summary on test methods, applicable formation and references

Test method	Applicable formation	Reference
Various Field Methods	Soil & Rock Aquifers	ASTM D 4043
Pumping tests	Drawdown in soils	ASTM D 4050
Double-ring infiltrometer	Surface fill soils	ASTM D 3385
Infiltrometer with sealed ring	Surface soils	ASTM D 5093
Various field methods	Soils in vadose zone	ASTM D 5126
Slug tests.	Soils at depth	ASTM D 4044
Hydraulic fracturing	Rock in-situ	ASTM D 4645
Constant head injection	Low-permeability rocks	ASTM D 4630
Pressure pulse technique	Low-permeability rocks	ASTM D 4630
Piezocone dissipation	Low to medium k soils	Houlsby&Teh (1988)
Dilatometer dissipation	Low to medium k soils	Robertson et al. (1988)

Variable head tests

Seepage tests in boreholes constitute one means of determining the in-situ permeability. They are valuable in the case of materials such as sands or gravels because undisturbed samples of these materials for laboratory permeability testing are difficult or impossible to obtain. Three types of tests are in common use: falling head, rising head, and constant water level methods.

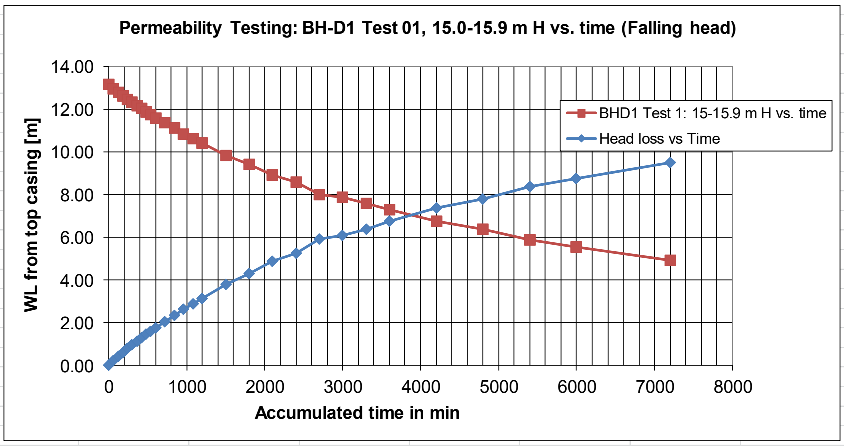
In general, either the rising or the falling level methods should be used if the permeability is low enough to permit accurate determination of the water level. In the falling level test, the flow is from the hole to the surrounding soil and there is danger of clogging of the soil pores by sediment in the test water used. This danger does not exist in the rising level test, where water flows from the surrounding soil to the hole, but there is the danger of the soil at the bottom of the hole becoming loosened or quick if too great a gradient is imposed at the bottom of the hole. If the rising level is used, the test should be followed by sounding of the base of the hole with drill rods to determine whether heaving of the bottom has occurred. The rising level test is the preferred test. In those cases where the permeability is so high as to preclude accurate measurement of the rising or falling water level, the constant level test is used.Holes in which seepage tests are to be performed should be drilled using only clear water as the drilling fluid. This precludes the formation of a mud cake on the walls of the hole or clogging of the pores of the soil by drilling mud. The tests are performed intermittently as the borehole is advanced. When the hole reaches the level at which a test is desired, the hole is cleaned and flushed using clear water pumped through a drill tool with shielded or upward-deflected jets. Flushing is continued until a clean surface of undisturbed material exists at the bottom of the hole.

Table 2-8: Typical falling head format and example of real test result

Permeability testing - Falling Head Method

Project	Beko Abo MPP	Consultant	NORPLAN
Site	Top Right bank	Drilling Contractor	RODIO - ARDCO
Borehole No.	**BH-D1**	Test Sectionr	15.0 -15.93
Borehole Depth b.GL. (m)	15.93	Date	06 06 11
Water Level Before (m)	14.6	Casing inner dia. [m]	0.0651
Top of casing above GL(m)	0.58	Rod size Inner dia	0.08
Ground water level below GL (m)	14.6	BH dia. D [m]	0.098
Casing Depth b.GL. (m)	15	Time started	03:50
Length of test section L=	0.93	Area of water column: A=Pi*d*d*/4 [m2]	0.003328525
L/D=	9.489795918	Form factor F=Pi*2*L/ln((/L/D+ROT(1+(L/D)*(L/D)))	1.98
		k= log(H1/H2)*A/(F*(T2-T1))	
Inclination measured from horizontal	60		
Sinus to inclination	0.8660		

Reading	Elapsed Time (min.)	Water level form top	Δ head (cm)	Delta head in m	Time sec T	H in m, measuerd along the borehole	H Vertical	log(H1/H2)	k [m/s]
1	0	0	0		0	15.18	13.15		
2	1	0.22	22.0	0.22	60	14.96	12.96	0.014598799	4.1E-07
3	2	0.42	20.0	0.2	120	14.76	12.78	0.013459153	3.8E-07
4	3	0.6	18.0	0.18	180	14.58	12.63	0.012270093	3.4E-07
5	4	0.79	19.0	0.19	240	14.39	12.46	0.013117206	3.7E-07
6	5	0.96	17.0	0.17	300	14.22	12.31	0.011884097	3.3E-07
7	6	1.13	17.0	0.17	360	14.05	12.17	0.012027029	3.4E-07
8	7	1.29	16.0	0.16	420	13.89	12.03	0.011453239	3.2E-07
9	8	1.46	17.0	0.17	480	13.72	11.88	0.012314534	3.4E-07
10	9	1.6	14.0	0.14	540	13.58	11.76	0.0102565	2.9E-07
11	10	1.78	18.0	0.18	600	13.4	11.60	0.013343415	3.7E-07
12	12	2.06	28.0	0.28	720	13.12	11.36	0.021116923	3.0E-07
13	14	2.35	29.0	0.29	840	12.83	11.11	0.022351605	3.1E-07
14	16	2.65	30.0	0.3	960	12.53	10.85	0.02366041	3.3E-07
15	18	2.9	25.0	0.25	1080	12.28	10.63	0.020153846	2.8E-07
16	20	3.14	24.0	0.24	1200	12.04	10.43	0.019737483	2.8E-07
17	25	3.8	66.0	0.66	1500	11.38	9.86	0.056377011	3.2E-07
18	30	4.3	50.0	0.5	1800	10.88	9.42	0.044931187	2.5E-07
19	35	4.87	57.0	0.57	2100	10.31	8.93	0.053811943	3.0E-07
20	40	5.24	37.0	0.37	2400	9.94	8.61	0.036547277	2.0E-07
21	45	5.94	70.0	0.7	2700	9.24	8.00	0.073025135	4.1E-07
22	50	6.1	16.0	0.16	3000	9.08	7.86	0.017467693	9.8E-08
23	55	6.4	30.0	0.3	3300	8.78	7.60	0.033597785	1.9E-07
24	60	6.75	35.0	0.35	3600	8.43	7.30	0.040679636	2.3E-07
25	70	7.37	62.0	0.62	4200	7.81	6.76	0.076391808	2.1E-07
26	80	7.8	43.0	0.43	4800	7.38	6.39	0.056631325	1.6E-07
27	90	8.37	57.0	0.57	5400	6.81	5.90	0.080381518	2.2E-07
28	100	8.75	38.0	0.38	6000	6.43	5.57	0.057417582	1.6E-07
29	120	9.5	310.0	3.1	7200	5.68	4.92	0.124023306	5.3E-08
					Average Permeability				2.8E-07



The permeability is then determined by one of the procedures given below. Specifications sometimes require a limited advancement of the borehole without casing upon completion of the first test at a given level, followed by cleaning, flushing, and repeat testing. The difficulty of obtaining satisfactory in situ permeability measurements makes this requirement a desirable feature since it permits verification of the test results.

Data which must be recorded for each test regardless of the type of test performed include:

- Depth from the ground surface to groundwater surface both before and after the test,
- Inside diameter of the casing,
- Height of the casing above the ground surface,
- Length of casing at the test section,
- Diameter of the borehole below the casing,
- Depth to the bottom of the boring from the top of the casing,
- Depth to the standing water level from the top of the casing, and
- A description of the material tested.

Falling water level method

In this test, the casing is filled with water, which is then allowed to seep into the soil. The rate of drop of the water surface in the casing is observed by measuring the depth of the water surface below the top of the casing at 1, 2 and 5 minutes after the start of the test and at 5-minute intervals thereafter. These observations are made until the rate of drop becomes negligible or until sufficient readings have been obtained to satisfactorily determine the permeability. Other required observations are listed above.

K= A/(FT) or $K = \frac{A}{F(t2-t1)} ln(H1/H2)$...(2.3)

Where:

- A = cross sectional area of casing
- F= Intake factor
- H1 &H2 = the variable heads measured at t1 & t2
- T = the basic time factor

Table 2-9: Range of hydraulic conductivity

Formation	Coefficient of permeability, K (cm/s)	Degree of permeability
Gravel	$>10^{-1}$	Very high
Sandy gravel, clean sand, and fine sand	10^{-1} to 10^{-3}	High to Medium
Sand, dirty sand, silty sand	10^{-3} to 10^{-5}	Low
Silt, silty clay	10^{-5} to 10^{-7}	Very Low
Clay	$>10^{-7}$	Impermeable

Rising water level method

This method, most commonly referred to as the “time lag method” (US Army Corps of Engineers, 1951), consists of bailing the water out of the casing and observing the rate of rise of the water level in the casing at intervals until the rise in the water level becomes negligible. The rate is observed by measuring the elapsed time and the depth of the water surface below the top of the casing. The intervals at which the readings are required will vary somewhat with the permeability of the soil. The readings should be frequent enough to establish the equalization diagram. In no case should the total elapsed time for the readings be less than 5 minutes. As noted above, a rising level test should always be followed by a sounding of the bottom of the hole to determine whether the test created a quick condition.

Constant water level method

In this method water is added to the casing at a rate sufficient to maintain a constant water level at or near the top of the casing for a period of not less than 10 minutes. The water may be added by pouring from calibrated containers or by pumping through a water meter. In addition to the data listed in the above general discussion, the data recorded should consist of the amount of water added to the casing at 5 minutes after the start of the test, and at 5-minute intervals thereafter until the amount of added water becomes constant.

$$K = \frac{Q}{5.5RH} \quad \text{……………………………………………………………………… (2.4)}$$

Where:

K = Permeability value at field temperature

Q = Rate of follow in to hole

R = internal radius of casing

H = Differential head of water (H -H_f)

2.3.4 Geophysical survey

For areas with complex geological conditions, conducting geophysical survey is highly helpful in determination of depth and variations of the rock depth along the axis of divession structures foundation. It will also support information obtained by other methods. A number of geophysical methods are used in preliminary investigation of sub-soil strata. The methods can be used for the location of different strata and for a rapid evaluation of the sub-soil characteristics. However, these methods are very approximate. The geophysical methods frequently used in geotechnical investigation can be broadly divided into the two categories: Seismic methods and Electrical resistivity methods as described in the following sections. Selection is made based on nature of the engineering problem, available geological information, and site-specific criterion such as the depth of the target, required resolution, site accessibility, and cost. Manuals such as BS 5930, 1999, ASTM D6429, and ASTM D5753 could be referred to select suitable methods of geophysical investigation. Detail of each methods and their advantage and disadvantages is presented in section 3.2.6 of this guid line.

2.3.5 Explatory borehole

Depending on the size of the diversion structure and the complexity of the site, drilling of boreholes with propore insitu tests is may be required to cleary understand the site. Number and deepths of these explatory investigation boreholes for the diversion structure site has to be decided following the results of the investigation methods discussed above. There are two basic types of rotary drilling: open hole (or full hole) drilling, where the drill bit cuts all the material within the diameter of the borehole; and core drilling, where an annular bit, fixed to the bottom of the outer rotating tube of a core barrel, cuts a core, which is recovered within the innermost tube of the core barrel assembly and brought to the surface for examination and testing. Rotary drilling for ground investigation is usually core drilling. When open hole drilling or coring, temporary casing is normally used to support unstable ground or to seal off fissures or voids, which cause excessive loss of drilling fluid. Drilling fluid additives or cement grouting may sometimes be satisfactory alternatives. The rotary drilling rig should be well maintained and should be capable both of controlling rotational speed and providing axial load and torque to suit the nature and hardness of the material penetrated, the diameter of the core barrel and drill string, drilling fluid and flushing system, weight of drill string and installation of temporary casing(s). Detail of each methods and their advantage and disadvantages is presented in section 3.2.7 of this guid line.

2.3.6 Determination of rebound hammer strength of rock

In diversion structures foundation sites, rocks are found exposed with varying strength, weathering grade and engineering characteristic. Excavating test pits manually is not practical and mobilizing drilling rig and the crew may not be economical. In such situations, rebound value of the Schmidt Hammer could be used as an index value for the intact strength of rock materials, but it is also used to give an indication of compressive strength of rock material. The Schmidt hammer is essentially a field instrument but it may be used as well in the laboratory. When used in the laboratory special attention has to be given to the connection of the sample with the V-block and the base, and to the connection with the supporting table, as these factors have a great influence on the testing results. The method is of limited use on very soft and on very hard rocks.

Apparatus used

- A standard Schmidt hammer preferably the L-type having an impact energy of 0.74NM
- A calibration test anvil for the calibration of the test hammer rebound numbers.
- A steel base of minimum weight of 20 KG with a steel V-block for cyllenderical samples and with clamping device. Such a base can also be used if irregularly formed samples (lumps) should be tested in the laboratory.
- A vertically oriented cradle to guide the Schmidt hammer in a vertical downward direction during the testing

Test Procedures and considerations

1. Prior to the testing sequence, the Schmidt hammer should be calibrated using the calibration anvil supplied by the manufacturer of the Schmidt hammer for the purpose. The average value of 10 readings on the test anvil should be obtained.
2. The specimen obtained for the laboratory testing should be representative and characteristic for rock material to be studied. The L-type hammer should be used on NX (>/= 54.7mm) or larger diameter core samples preferably T2 (>/=84mm) size for N type hammer or on block samples having a thickness of at least 100mm at the point of impact. the specimens must be securely clamped to a ridge base to adequately secure the specimen against vibration and movement during the test. The base must be placed on a flat surface that provide firm support
3. The test surface of all specimens in the laboratory and testing locations in the field should be smooth and flat over the area of contact with plunger. This area and the rock material beneath the surface to a depth of 60mm shall be free from cracks, or any discontinuity in the rock.
4. The hardness values obtained will be affected by the orientation of the hammer. In the laboratory the vertical cradle will hold the Schmidt hammer in vertical downwards position during the testing. During insitu testing in the field the testing direction must always to the surface tested. The orientation of the Schmidt hammer in that case should be recorded and reported in the results. With the Schmidt hammer a chart is provided to make corrections for non-vertical measurements.
5. At least 20 individual tests must be conducted on any rock sample. The test locations shall be separated by at least the diameter of the plunger. Any test that causes cracking or any other visible failure of the rock should be rejected. Errors in specimen preparation and testing tend to produce low hardness values.

Calculation

a. No correction is required if the Schmidt hammer reading on the anvil gives values :
   i. For L type between 72 and and 76
   ii. For N type between 78 and 82

   The correction formula can be used down to a rebound number on the anvil: for L type 72, for N-type 78. If the rebound on the anvil is lower the schmiidt hammer must be cleaned.

   The correction value is calculated as follows

   $$correction\ factor = \frac{\text{specified standard values of the anvil}}{\text{average of 10 readings on the anvil}},$$

   Standard value on the anvil: 74 for L-type and 80 for N type

b. For tests in the laboratory on rock materials of uniform strength the measured test values the sample must be ranked in a descending value. The lower 50% of the values should be discarded and the average calculated of the upper 50% values. This average shall be multiplied by correction factor to obtain the Schmidt rebound hardness. Preferably we use L- type Schmidt hammer for rock testing however if you only have a N-type, you have to convert the N reading in to a L- reading.

   i. <30 = N reading -6
   ii. 30 – 50 = N reading -5
   iii. > 50 = N reading -4

c. When a number (15 to 20) reading is taken in the field to characterize a unit with some variation of hardness and strength, the mean of the rebound values is then computed and checked for "off-shots". The collected data is then analyzed for obvious "off-shots", that is, those rebound readings that deviate from the mean by more than six units (BS EN 12504-2:2001). In this case the reading should be eliminated and/or additional reading could be taken by further impact test and then averaged is calculated.

   The graph provided from the manufacturer of the Schmidt hammer is used to read the compressive strength as a function of the average rebound values. The correction for non-horizontal impact direction is also corrected based on the graph given by the manufacturer.

   The corrected rebound values of each test location are summed up and their average computed; this average value is used to evaluate the most likely compressive strength value, from a table/graph correlating rebound number with strength supplied by the manufacturer.

Remarks
** When measurements are taken insitu, the testing location is usually not as flat as is required. By repetition of the test several times at exactly the same location the rebound value usually is observed to increase until a constant value is reached. This is due to cracking of points of the surface asperities and subsequent increase of the contact area between the plunger and the rock surface. The reading must be taken after it has reached a stable value.

Example: To calculate the compressive strength for the following 10 readings:

Readings: 41, 30, 40, 47, 42, 49, 38, 46, 36, & 38

Average = 40.7 ≈ 41

Expected maximum = 41+6 =47; and expected minimum = 41-6 = 36.

However, as we can see from the list of reading data, there exist two readings (30 & 49), one is below the minimum and one is above the maximum. Both of them should be removed and the remainings have to be averaged. As the result, the rebound value for this rock exposure is 41. Then corrected using correction factor obtained from anvil will be applied and read the compressive strength value from the graph following the direction of application.

Reporting

The report shall include the following information:

a. Data on sampling
   i. Project name, location, date of sampling, sampling number, depth below the ground level (in case of borehole)
   ii. Type of sample, (core, block, broken, insitu or other), sample dimensions
   iii. Lithology, weathering grade, grain size, natural water content
   iv. Sample transport and storage conditions
b. Data on specimens
   i. Form, dimensions and weight of all specimens
   ii. Density and water content during testing
c. Data on testing procedure
   i. Type of Schmidt hammer
   ii. Orientation of the Schmidt hammer during every test in the field
   iii. Method of clamping used in the laboratory
   iv. The Schmidt rebound hardness value as obtained from the calculations discussed above.
   v. Indication of the value of uniaxial compressive strength in Mpa +/- the average dispersion as read from the graph, with reference to the fact that the graph has been used for this purpose. The rock density must be known when this graph is used.

Table 2-10: suggested format for collection of rebound hammer test data in the field for rock mass characterization

Nr.: ↓ ↖ ↑ ↘ ↙	Nr.: ↓ ↖ ↑ ↘ ↙	Nr.: ↓ ↖ ↑ ↘ ↙	☐ Type N ☐ Type L
Nr.: ↓ ↖ ↑ ↘ ↙	Nr.: ↓ ↖ ↑ ↘ ↙	Nr.: ↓ ↖ ↑ ↘ ↙	Description of the rock materail
Nr.: ↓ ↖ ↑ ↘ ↙	Nr.: ↓ ↖ ↑ ↘ ↙	Nr.: ↓ ↖ ↑ ↘ ↙	

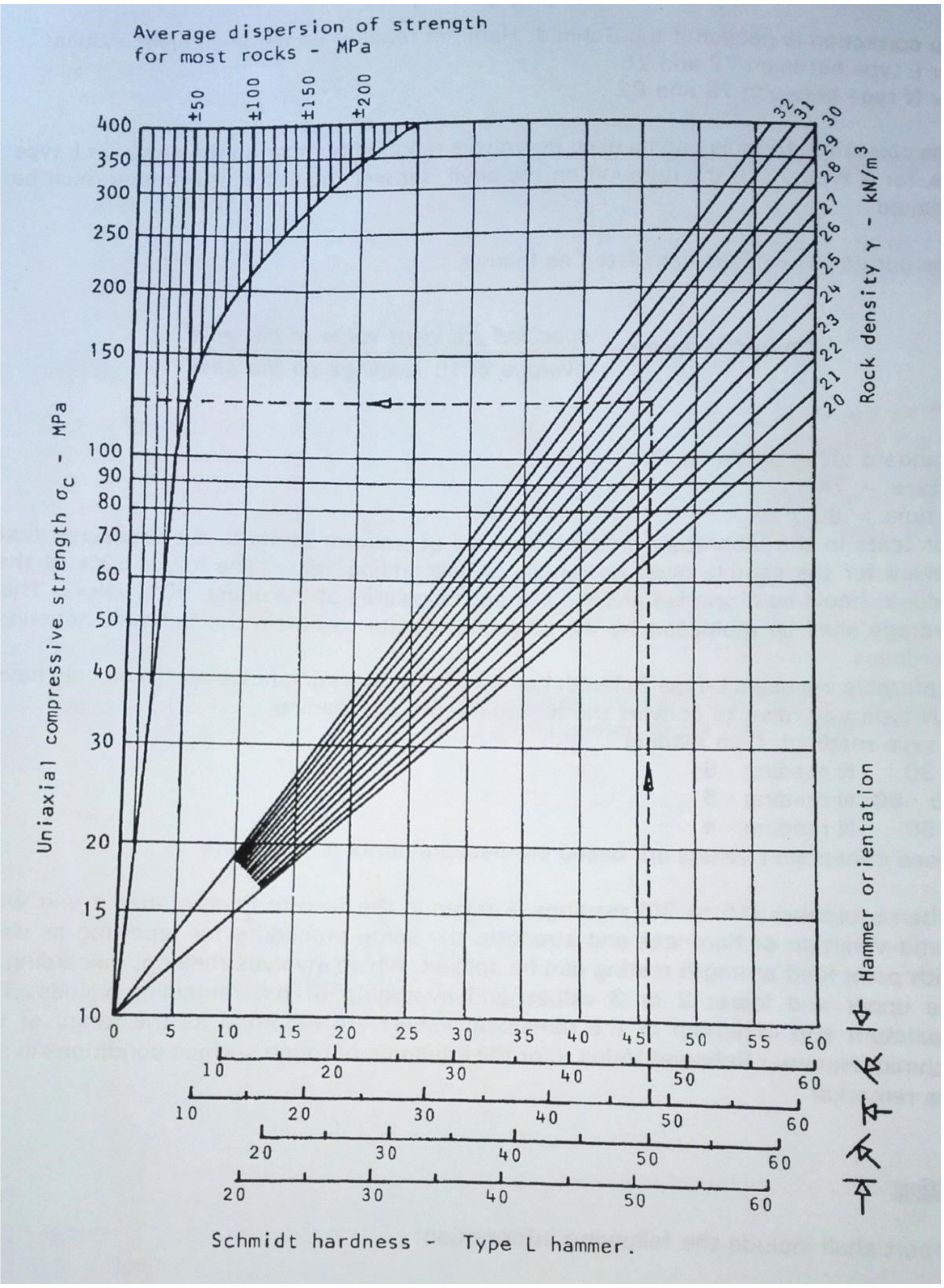


Figure 2-7: Relationship between the schmidt hammer rebound hardness (type L hammer) and uniaxial compressive strength of rock (ISRM).

2.3.7 Assessment of rock excavatability

Excavatability of rock depends on the geotechnical properties of the material, on the method of working, and on the type and size of excavation equipment to be used. It is generally accepted that discontinuity spacing and the strength of the intact rock are particularly important properties. The aperture, infilling and the wall strength of the discontinuities are also important factors. The main excavation methods are blasting, ripping and digging. A number of methods are suggested in the literature to assess the rock excavatability. Each system considers a different set of geotechnical parameters.

The graph suggested by Franklin et al. (1971) considered only two parameters: fracture (joint) spacing index, I_f, and point load strength index, I_s. Ripper performance charts published by Caterpillar (1988) considered only seismic velocity. Weaver (1975) proposed a 'rip ability rating chart'. This chart was adapted from the Rock Mass Rating (RMR) system used for tunnel support design (Bieniawski, 1974). The main changes were the replacement of Rock Quality Designation (RQD) with seismic velocity, the introduction of a weathering parameter and adjustments for the effects of discontinuity orientation.

Kirsten (1982) proposed an 'excavatability index', N, based on the Q-system for tunneling (Barton et al., 1974). He also suggested adjustment for discontinuity orientation in ripping. Minty and Kearns (1983) modified Weaver's rip ability rating chart and suggested a 'geological factors rating' (GFR) which considers groundwater condition and surface roughness of discontinuities. Scoble and Muftuoglu (1984) devised a 'digability index' based on discontinuity spacing, rock strength and weathering. Smith (1986) suggested that the seismic velocity proposed by Minty and Kearns (1983) should be omitted in the evolution. Singh et al. (1987) developed an alternative rip ability rating chart. They considered seismic velocity, point load strength index, weathering and discontinuity spacing. Ripper performance charts published in the Caterpillar Performance Handbook (Caterpillar, 1988). These charts consider only seismic velocity of various rock types for assessment of rock excavatability. Karpuz et al. (1990) modified the graph suggested by Franklin et al. (1971) and considered seismic velocity, unconfined compressive strength of intact rock, rock hardness, weathering and discontinuity spacing. Hadjigeorgiou and Scoble (1990) also considered point load strength, weathering, discontinuity spacing and discontinuity orientation in their assessment of excavatability. Kentli and Topal (2004) used to the chart of excavatability for rock suggested by Pettifer and Fookes (1994) and suggested that spacing of rock joints at the deeper levels is also considered due to joint spacing may be increased.

The revised and current excavatability chart proposed by Pettifer and Fookes (1994) under use for the assessment of rock excavatability in different hydraulic as well as other infrastructures construction site for assessement and tendering. The excavatability chart considers the types of excavation equipment and requires engineering geological parameters such as the discontinuity spacing index (I_f) and point load strength index ($I_{s(50)}$). These parameters are relatively easy to obtain through field and laboratory studies. Joint spacing could be measured at the site and discontinuity spacing index (I_f) was calculated from the following equation by the ISRM (1981):

$$I_f = \frac{3}{J_v}$$

where J_V is volumetric joint count and it was calculated from the following equation suggested by the ISRM (1981):

$$J_v = \frac{1}{S_1} + \frac{1}{S_2} + \frac{1}{S_3}$$

where S_1, S_2 and S_3 are discontinuity spacing of joint sets.

The point load strength index, $I_{s(50)}$, is established through collecting sufficient samples of rock from site under investigation and tested in accordance with the ISRM method (ISRM, 1985), and their values were used to determine for excavatability of the site. Examples for the parameters used are presented in table 2.11 below.

The plotting of the data in the revised excavatability chart for the following two data used as an example is shown in fig. 2.6. Based on the excavatability analysis of the data used as an example, the excavatability category at the right slope is generally easy ripping. However, it is hard ripping and easy ripping for the left slopes of the dam site from which these data collected.

Mean discountinuty spacing, discontinuty spacing index I_f, point load strength index $I_{s(50)}$ and excavatablity classes of right and left slope (real data of dam site is used as an example for determination how to evaluate) shown in Table 2-11

Table 2-11: mean discountinuty spacing, discontinuty spacing index I_f, point load strength index $I_{s(50)}$

Slope/exposure	Mean discontinuity spacing(m)			Discontinuity	Point load	excavatablity
	J-1	J-2	J-3	Spacing Index I_f	Strength Index	Class
Right	0.046	0.068	0.980	0.080	0.71-7.98	Easy ripping
Left	0.075	0.095	0.146	0.098	1.94-9.71	Easy hard ripping

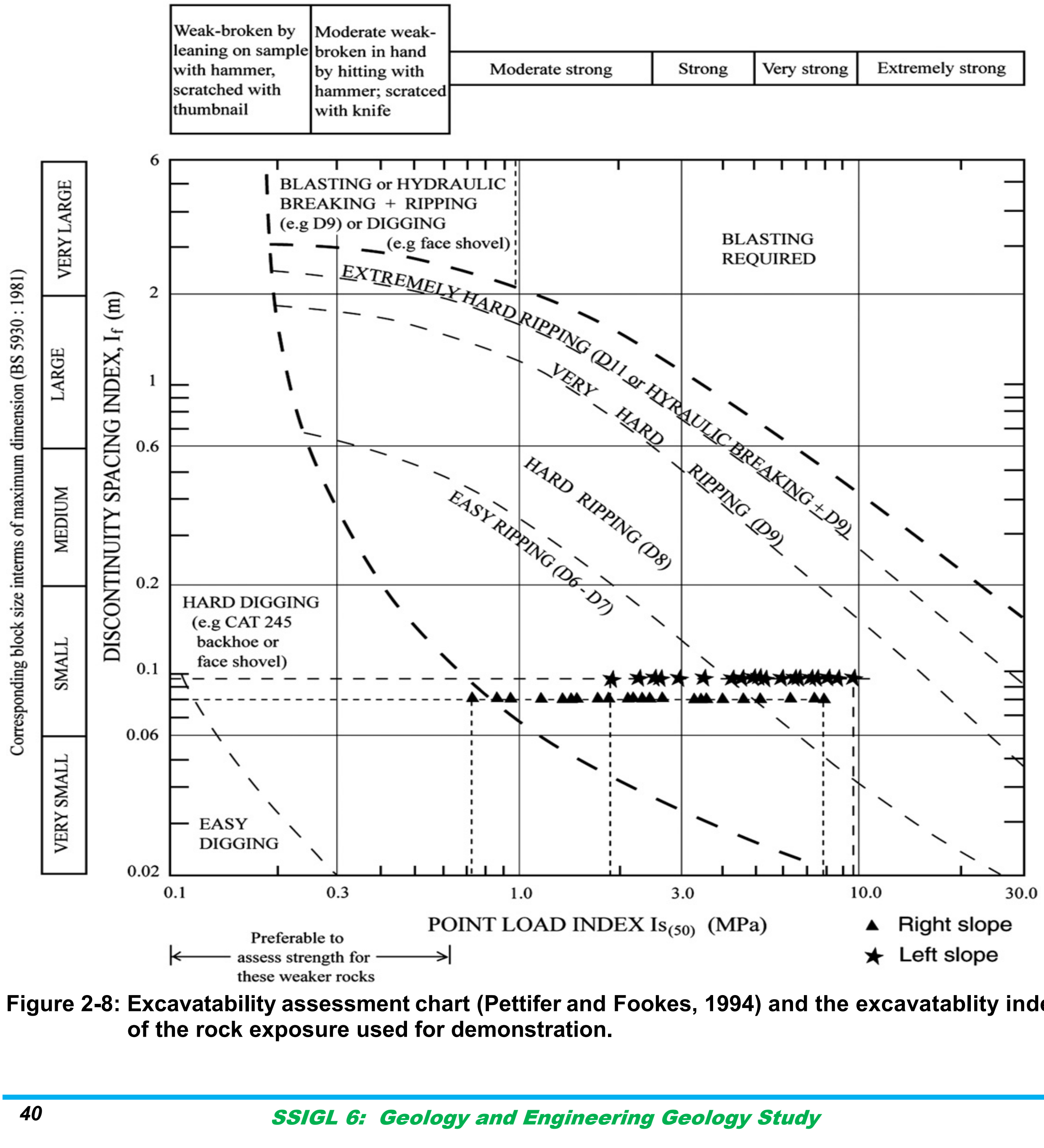


Figure 2-8: Excavatability assessment chart (Pettifer and Fookes, 1994) and the excavatablity index of the rock exposure used for demonstration.

The excavatability analysis reveals that the excavatability of the data used as an example ranges from hard digging to hard ripping for the right slope. The excavatability for the left slope is generally easy ripping. However, these excavatability ranges are valid for surface or upper levels. Due to weathering and surface conditions, the joint spacing and point load index values at the upper levels may be less than the values at the deeper levels. Thus, it was obtained from a drilling core investigation that the joint spacing decreases at the deeper levels. As a result of this, excavatability of this site we use as an example ranges from hard digging to hard ripping for the right and left slope at the deeper levels.

2.3.8 Summary of site investigation in terms of each diversion and associated structures type

General

Site investigation for geological and geotechnical investigation vary in type of the investigation as well as intensity or volume of work as per the structures planned to be constructed and the complexity of the site and the stage of study undertaken.

As discussed in the part of planning of this manual, for a site investigation to be successful it has to be planned in orderly manner using appropriate field and laboratory equipment operated by experienced and skilled personnel.

Since the site complexity could not be standardized, as it varies from place to place, however considering the type of structures and stages of study, the following investigation types are foreseen. Summarized recommendations of site investigation are presented on the table 2-12 below.

Table 2-12: proposed location, spacing, depths of exploratory boreholes for different hydraulic structure

Structure type	Location of the structure's part	Investigation methods	Spacing (test pits) and features to be considered	Depth of investigation	remarks
Diversion structure	Axis	- excavating test pits - geophysical survey - explatory borehole drilling	- At least 3 TP excavation(at center, abutments) - Imaging method of VES & Seismic refraction using sladge hammer - At least 3 short boreholes drilling(at center, abutments)	Excavating TP up to bed rock or 5m max. -Cover the whole diversion structure alignment - estimated borehole depth 7 to 15m	Depending on depth of seepage
	Upstream	River bank stability assessment	1:1,000 engineering geological mapping	Up to the influence length of back water	Based on surveyed topo map.
Canal		- Engineering geological mapping and test pitting - Excavatability survey	TP excavations every 250m permeability tests every 500m	Up to half meter below the designed	Based on the formation type the number of Test pits

Structure type	Location of the structure's part	Investigation methods	Spacing (test pits) and features to be considered	Depth of investigation	remarks
				canal bed level	and permeability test could be varied.
Pond		Engineering geological mapping and test pitting	- TP excavations and conducting permeability test atleast two points - DCP test	Up to half meter below the depth of maximum cut	1:10,000 engineering geological mapping
Crossing structures		Engineering geological mapping and test pitting - Excavatability survey	- TP excavation with in situ - DCP test	Up to the bed rock or At least 3m below the existing surface	1:10,000 engineering geological mapping Checking bank stability and scouring effect
Borrow areas		TP excavation and auguring - Excavatability survey	TP excavation every 150 m by 150m grid	At least 3m below the existing surface	1:10,000 engineering geological mapping

2.4 CONSTRUCTION MATERIALS INVESTIGATIONS

2.4.1 General considerations

In geotechnical explorations, construction materials investigations comprise visual, field and laboratory assessments of locally available soil and rock materials. The assessments include impervious fills, embankment fills, transitions and drains, concrete ingredients, embankment protections, etc., of waterworks or other structures. Adequate coverage survey shall be carried out at the proposed site for identification of suitable sites for construction material. This shall cover:

Investigation for identification of locations of potential quarries for construction materials like rock, aggregates, sand, soils and water; and proximity, quality, ownership of the land where the material is found, accessibility situation, environmental and social impact of quarrying and its mitigation measures as well as hauling distance including access route are to be examined;

- Estimation of quantity of material for each location, details of sample collection/testing of the materials, quality/suitability of the material, road maps showing the transport road up to the borrow area in relation to the construction site(s) shall be provided;
- Identifying the borrow areas; and preparation of location maps, road maps etc. showing the transport road up to the borrow area, relating the same to the construction site(s);
- Collection and evaluation of samples from borrow areas (rock, sand, soil, aggregate and water) for its suitability for different types of materials shall be collected for laboratory tests;
- The depth of the pits/auger holes shall depend upon the availability of the soils and economic exploitation. The borrow area shall be located at near the working site as possible. Pits/auger holes (diameter 15 to 30 cm) shall be taken in the proposed borrow area on 30 to 50 m grid depending on the uniformity of soil depth & type and representative samples collected/tested for different types of strata/soil to determine their properties and delineate the soil zones;
- For assessment of quantities, drill holes shall be taken depending on the magnitude and type of the anticipated hydraulic structure;

- Required details of any other material as indicated in the earlier items shall be indicated.

2.4.2 Impervious materials

2.4.2.1 Properties required

Impervious materials are used in many parts of hydraulic structures such as in canals, ponds, etc. These impervious materials include clay soils, stabilized soils and concrete. In clay soils, there are desirable and non-desirable properties which the investigation has to look for use as impervious material for blanket, canal lining, etc. These objectionable properties, after compaction, include:

- High shrinkage and swelling,
- Perviousness,
- Compressibility,
- Erodibility,
- Non workability as construction materials,
- Low shear strength, etc.

These non-desirable properties can also be reduced by additives, blending, high quantities, etc., but always the economics have to be there.

2.4.2.2 Clay for fill and blanket

The impervious shall extend to full impervious depth or partially depending upon the extent of tolerable seepage allowed and other supplementary seepage prevention provided. Imperviousness, low shrinkage and swelling, good workability and low compressibility are highly needed.

As blanket, clays provide reduction of seepage loss, decrease in hydraulic gradient and uplifts to the structure.

Peat, organic clay and inorganic clays of high plasticity shall as much as possible be avoided for fill and blanket uses. Sources are from nearby excavations for appurtenance structures or borrow area of minimum distance from the structure.

2.4.2.3 Canal lining and blanket

On canal construction, linings are used to reduce water loss which in turn is required to conserve precious water, prevent water logging of adjacent lands and reduce the sizes of conveyance and headwork system.

Commonly used canal linings are clay, concrete, masonry, brick, etc. However, unless high degree of seepage control is required, clay lining is the most commonly used and locally available economic material.

For clay canal lining, high degree of impermeability, low shrinkage and swelling potential, erosion resistance and good workability are required. To achieve these properties thick linings, blending of good clays with silt, sand and gravel, and protective covers are needed. For this purpose, the sources are from canal excavation or nearby borrow area. Undesirable stones and plant growths shall be removed from the stockpile.

Before placing the lining materials on the bed and slopes of the canals, field and laboratory explorations and foundation preparations (excavation, scarifying and moistening) are required. Then blending (if necessary) and compaction in lifts at optimum moisture content are needed.

2.4.2.4 Canal fill

Depending upon the design requirements, earth embankments for main canals and laterals may consist of impervious or pervious soils placed loose, partially compacted by equipment or well compacted by rollers, or a combination of these. The selection criteria as earth embankment fill are not as such critical as that of linings, cores or blankets. Suitable material from canal excavation or borrow areas can be used. Usually cut and fill method with good compaction and lining is used.

2.4.3 Concrete aggregates

Aggregates are those chemically inert materials which when bonded with cement paste form concrete. They constitute the bulk of the total volume of concrete and hence they influence the strength of concrete hydraulic structures to a great extent. The aggregates shall be hard, strong, dense, durable, and free from injurious amounts of clay, loam, vegetable and other foreign matters. Depending upon their size, the aggregates are classified as fine aggregate (sand) and coarse aggregate (gravel).

2.4.3.1 Sand

Sand may be obtained from river, lake, sea and pits. The sum of all types of deleterious materials in fine aggregate shall not exceed 5%. River sand usually suitable for concrete but mostly contaminated with mud and thus is advisable to wash such sand before use. Sea sand is also contaminated with salts from sea water which are liable to attack the steel reinforcement. Pit sand also generally suitable, but it is liable to contain silt or other organic matter. The other sand source is by crushing suitable hard rock to the required grading. There is a better control and qualities from this source. Therefore the economic comparison must be made between the naturally available sands in the locality and from crushing before deciding the source. The characteristic requirements of sand for hydraulic structure are more or less similar to other concrete works. Generally more exacting investigations are needed than for other purposes.

Angular sand has good interlocking property which results in a strong mortar while rounded grained sand does not afford sufficient interlock in the matrix. Soundness values of less than 8% in five reaction cycles are assumed to be satisfactory.

The fineness modulus of sand, which is the sum of the cumulative percentage retained on the number 4, 8, 16, 30, 50 &100 US standard sieves divided by 100, shall, for general concrete work, be maintained between 2.50 to 3.00, and the variation from the average on any job shall be held ± 0.1 if uniformity and close control of placement are desirable [Creager, et al, 1995].

The grading of sand can be divided into four zones. The grading becomes progressively finer from grading zone I (coarse sands) to grading zone IV (fine sands). Sands of grading zones I to III are mostly used for plain and reinforced concrete but IV is normally restricted for special mix of concrete.Gravel

Gravels can be obtained from alluvial deposits or from crushed hard stones. The sum of the percentage of all type of deleterious substances in gravel shall not also, as sand, exceed 5%. As far as possible, flaky and elongated gravel shall be avoided. Soundness result (sodium sulphate test) shall be 10% for five cycles.

Gravels are usually obtained from crushed basalt, granite, gneiss and other suitable hard, dense and durable rocks.

In most cases of gravel, the material brought at site may be single sized aggregates (i.e.,ungraded) of nominal sizes 63 mm, 40mm, 20mm, l6mm, 12. 5mm and 10mm or it may be in the form of graded aggregates of nominal sizes 40mm, 20mm, I 6mm and 12. 5mm. See Table 8.2. The other is an all-in-aggregate (i.e., mixture of sand and gravel).

The size of the gravel used depends upon the nature of the work. The maximum size for mass concrete (as in dams) is 20mm and 63mm for plain concrete. For reinforced concrete, gravels having a nominal size of 20mm are generally considered satisfactory.

2.4.4 Rock sources

Rocks for constructions may be required as massive rocks for diversion structures, stilling basins and canals, drainage blankets and as masonry works in hydraulic structures. Rock sources shall satisfy two main criteria: first the source shall able to produce rock fragments in suitable sizes, i.e., moderate to slightly weathering and joints must exist for production and excavatability; secondly, the rock material shall be hard, dense and durable to withstand destructive forces that encounter during placing or due to wave action or due to normal weathering. Details proceedures on consideration rocks and water as construction materials is presented on sun topic 3.4.4 and 3.4.5 below

2.5 SAMPLING AND LABORATORY TESTS

2.5.1 General

Laboratory tests are conducted on soil as well as rocks. The tests are generally varied based on the type and size of the structures and the geologic formations (soil and rock) in which the infrastructures are going to be built. In addition, the type of lab test parameters required will depend on the type of foundation and construction materials available in the project site.The purposes of laboratory testing on samples of soil and rock may be summarized as follows:

a) To describe and classify the samples which represent the actual materials on the site
b) To investigate the fundamental behavior of soil and rocks which determine engineering properties and appropriate method to be used in the analysis.
c) To obtain soil and rock parameters relevant to the technical objectives of the investigation

2.5.2 Selection of testing program

Laboratory testing of soils is a fundamental element of geotechnical engineering. The complexity of testing required for a particular project may range from a simple moisture content determination to specialized strength and stiffness testing. Since testing can be expensive and time consuming, the geotechnical engineer should recognize the project’s issues a head of time so as to optimize the testing program, particularly strength and consolidation testing. Each test, or series of tests, should address one or more of the purposes listed

A detailed description of the ground is essential before embarking on a program of soil or rock testing. After this, it is necessary to consider carefully to what use the data obtained from laboratory tests are to be put, whether the samples represent the conditions being investigated and whether the information can reasonably be expected to assist in the solution of the engineering problems concerned. Certain considerations regarding laboratory testing, such as when, how much, and what type, can only be decided by an experienced geotechnical engineer. The following minimal criteria should be considered while determining the scope of the laboratory testing program:

- Project type (foundation, embankment, rehabilitation, buildings, etc.
- Size of the project
- Loads to be imposed on the foundation soils
- Types of loads; (i.e., static, dynamic, etc.)
- Critical tolerances for the project (e.g., settlement limitations)
- Vertical and horizontal variations in the soil profile as determined from boring logs and visual identification of soil types in the laboratory
- Known or suspected peculiarities of soils at the project location (i.e., swelling soils, collapsible soils, organics, etc.)
- Presence of visually observed intrusions, slickensides, fissures, concretions, etc.

The selection of tests should be considered preliminary until the geotechnical engineer is satisfied that the test results are sufficient to develop reliable soil profiles and provide the soil parameters needed for design

The principal factors which should be taken into account are:

Quality management

Tests should be carried out in a laboratory with the appropriate technical capabilities (i.e. geotechnical, mineralogical, chemical).

The capabilities and quality of the testing laboratory should be approved by the Geotechnical advisor.

The laboratory should have an appropriate Quality Assurance system and, preferably, external accreditation.

The laboratory should have a full-time supervisor. The supervisor should be a geotechnical specialist with experience in soil testing to the level required for the particular tests.
Tests should be carried out by suitably qualified technicians with experience in the particular test, under the direction of the supervisor.

Relevance of test results

Some laboratory tests are suitable only for particular types of soil. Where, after a test has been carried out, it is found that the test was not relevant to the actual sample used; a suitable comment on the result should be made in the report of the laboratory tests.

All laboratory test results should be examined critically by the laboratory supervisor and by the geotechnical advisor. They should be examined to check the following:

a) That the results are reasonable, there are no gross errors and the results refer to the correct samples;
b) That the results from similar samples are consistent (i.e. results from samples with similar descriptions should be comparable);
c) That results from a particular sample are consistent (e.g. undrained strength should be consistent with the liquidity index);
d) That the results of the laboratory tests are consistent with the results of the in-situ tests;
e) That the test results are consistent with accepted data for similar soils and rocks whenever test results appear to be anomalous the reason should be established by re-examination of the original results or confirmed by further testing. If the reasons for a strange result cannot be established the anomaly should be clearly stated in the report on the laboratory tests

2.5.3 Sampling, transportation and storage of samples for laboratory tests

An important feature of a geotechnical laboratory is the provision of good facilities for handling, storing and inspecting samples. All samples entering the laboratory should be registered and receipts issued.

Handling and labeling

Note should be taken of any warnings on the labels of likely contamination. Samples should be treated with care to prevent damage, both during transportation and on arrival in the laboratory.

SAMPLE TAG

DATE ______________________________
PROJECT __________________________
CLIENT: ____________________________
Test Pit, Augur hole or Borehole ID:____________
Depth: ______________________________
Sample Type _______________________
Field Description ____________________
Tests to be conducted: 1. _____________________

Storage of samples

On arrival at the laboratory each sample should be registered. Samples should be stored so that they are protected from damage and deterioration, for example, from excessive heat or change in humidity, and so they can readily be located when required for examination or testing. Disturbed samples should be placed in purpose-made carriers that prevent loss of moisture content.

Tube samples and core samples should normally be stored on their sides in purpose-made racks with the possible exception of very soft marine sediment. The sample storage area should be of sufficient size to cater for the number of samples being handled without overcrowding. Ideally samples should be stored in a temperature and humidity controlled enclosure.

Inspection facilites

A specific area should be established in the laboratory for the inspection and description of samples. This should have sufficient space for the temporary stacking of the samples and an adequate area of bench space with good lighting, preferably daylight, for the actual inspection.

In general, the following equipment should be provided:

a) An extruder for removing samples from the sample tubes or liners and a means of splitting plastic liners when extrusion is not desirable;
b) An adequate number of trays to enable disturbed samples of granular soils to be tipped out for inspection, and some means of returning them quickly to their containers afterwards;
c) Spatulas and knives for splitting samples;
d) Dilute hydrochloric acid to assist identification of carbonate soils and rocks;
e) a water supply to enable the fines to be washed out of samples of soils and facilitate description of the coarser particles; to clean rock cores and block samples; and to wet-up fine grained soils;
f) A balance suitable for checking the adequacy of bulk samples for testing;
g) Means of resealing samples required for further use;
h) Washing facilities for personnel inspecting the samples so that they can keep themselves and their notes clean;
i) Hand lens, geological hammer, penknife, meter scale and protractor for logging cores;
j) A simple stereo microscope with magnification to times 30.

2.5.4 Visual examination and description of laboratory samples

Introduction

Description of samples of soil and rock tested in the laboratory forms an important part of the record of the test results. Such descriptions should be included on the laboratory work sheet(s). Descriptions of samples made in the laboratory should be compared with the equivalent field descriptions and any anomalies resolved.

Information about the grading and plasticity of soils can be estimated from inspection of bulk samples obtained during drilling, from tube samples not required for further intact testing or from trimmings from sample preparation operations.

Information about the structure and fabric of soils and the jointing in rocks can be obtained from inspection of high quality samples, but is often best carried out by inspection of the ground in test pits and exposures.

Variability, disturbance and representativeness of the samples are all important and appropriate comments should be made.

More detailed specialist examination of soil and rock samples may be valuable: these may include information of the paleontology and examinations of the fabric, texture and petrography by optical or electron microscopy.

Description of samples

All samples to be tested should be described prior to testing and the information recorded on the laboratory worksheet for the test. It would be good practice to select other representative samples for detailed description during laboratory testing.

Photographic records

Photographic records are a most valuable supplement to the written description or log.

Photographs can be used to provide a continuous record of, for example, rock cores, typical features, such as in split tube samples, or a record of atypical features. Where a continuous record is sought, common magnification, lighting and format should be used. All photographs, including those taken in the field should include a scale, color chart, details of the provenance of the sample and the site location. Color can be used to illustrate features clearly, but often monochrome can provide better definition of some features.

Natural light is preferable to artificial light, but in the laboratory this is often difficult; when light is poor careful use of flash can give good results.

Photographs can be taken of samples in the ‘as received’ state before testing and again after testing to show the mode of failure, or after splitting to show features of the fabric. Certain features may be seen more clearly after the sample has been allowed to dry or partially dry.

The photographic record of laboratory samples can be supplemented with photographs from the field showing natural or excavated faces in quarries or test pits spoil heaps and photographs of features observed during the walk over survey.

2.5.5 Tests on soil

Laboratory tests are used to determine design parameters and to complement field observations, field testing and back analysis of the behavior of existing structures that have been monitored. In some cases a field test may give more realistic results because of reduced problems of sample disturbance. However, there is a large body of practical experience behind most of the more common tests and when the data derived from them are used with skill and experience, reliable predictions usually result. In assessing the quality and relevance of laboratory test results the following points are considered.

Sample quality

It is essential that the sample used is of sufficiently high quality for the test in question. When samples arrive in the laboratory, all necessary steps should be taken to ensure that they are preserved and stored at constant temperature, humidity and natural moisture content and that they suffer the minimum amount of shock and disturbance.

Laboratory tests should be carried out at constant temperature, preferably in an air-conditioned laboratory.

Where the test is carried out on a nominally ‘undisturbed’ sample, the sample may, in truth, be far from undisturbed: indeed the act of taking the sample releases the initial state of stress in it. In preparing the laboratory test specimen, there is further disturbance and unavoidable change in the stress conditions. As a result, the test is not usually carried out at the same state of stress and water content as exist in the natural ground unless attempts are made to return the specimen to its in-situ state

Sample size

For disturbed samples, the quantity of soil required for any particular test is given in Table 2.28. As the behavior of the ground can be significantly affected by discontinuities, 'undisturbed' samples should ideally be sufficiently large to include a representative pattern of these discontinuities. This may require use of large 'undisturbed' samples.

Sampling of rock and soil

The frequency of sampling and testing in a borehole depends on the information that is already available about the ground conditions and the technical objectives of the investigation. In general, the field work covers three aspects, each of which may require a different sampling and testing program and may also require phasing of operations. These aspects are as follows:

a) The determination of the character and structure of all the strata and the ground water conditions;
b) The determination of the properties of the various strata whose locations have been determined in a), using techniques for sampling and testing that are normally available in routine ground investigations;
c) The use of special techniques of sampling and testing in strata where the normally available techniques have given, or may be expected to give, unsatisfactory results.

Sampling technique

The selection of sampling technique depends on the quality of the sample that is required and the character of the ground, particularly the extent to which it is disturbed by the sampling process. It should also be borne in mind that the behaviour of the ground in mass is often dictated by the presence of weaknesses and discontinuities. It is therefore possible to obtain a good sample of material that may be unrepresentative of the mass. In choosing a sampling method, it should be made clear whether it is the mass properties or the intact material properties of the ground that are to be determined

There are four main techniques for obtaining samples:

a) Taking disturbed samples from the drill tools or from excavating equipment in the course of boring or excavation
b) drive sampling, in which a tube or split tube sampler having a sharp cutting edge at its lower end is forced into the ground, either by a static thrust or by dynamic impact
c) Rotary sampling, in which a tube with a cutter at its lower end is rotated into the ground, thereby producing a core sample
d) Taking block samples specially cut by hand from a trial pit, shaft or heading Samples obtained by techniques b), c) and d) are often of sufficient quality to enable the ground structure within the sample to be examined. The quality of such samples can vary considerably, depending on the technique and the ground conditions, and most exhibit some degree of disturbance.

Table 2-13: Mass of soil samples required for laboratory tests

Purpose of sample	*Soil*	*Mass of sample required Kg*
Soil identification, including Atterberg limits; sieve analysis; moisture content and sulfate content tests	*Clay, silt, sand*	*1*
	Fine & med. Gravel	*5*
	Coarse gravel	*30*
Compaction test	*All*	*25 – 60*
Comprehensive examination of construction materials, including soil stabilization	Clay, Silt, Sand	100
	Fine and med. Gravel	130
	Coarse gravel	160

Conditions of test

Where, as in the case of measurements of soil strength and compressibility or stiffness, tests can be carried out under several different sets of conditions, the particular test should be selected and carefully specified by a geotechnical specialist with relevant experience.

The test carried out should be the one that determines the parameters required for the proposed design calculations. In many instances the test conditions and stress paths used should approximate to those that exist in the field at the time being considered in the design

Generally, laboratory tests in soil mechanics practice can group as

- Classification tests
- Tests to determine the strength properties of soil
- Tests to determine the compressibility of soil
- Tests to determine the compaction of soils
- Tests to determine the permeability of soils
- Chemical tests

Frequently needed and recommended laboratory test types along with the type of the structures with their parameters needed to be evaluated are presented below on the table 2-29. Detail proposes, summarized testing procedures, standards and remarks on each test types are presented on appendix-I.

Table 2-14: Summary of recommended laboratory tests along with the diversion structure / head work and associated structures of SSIP

Structure types	Engineering Evaluations	Required Information for Analyses	LaboratoryTesting
Diversion structure /Head work	- Bearing capacity - Depth to rock bed - Scouring problem - Bank stability	subsurface profile (soil, -ground water, rock) - compressibility parameters - shear strength parameters - geologic mapping including orientation and characteristics of rock discontinuities shrink/ swell/degradation of soil and rock fill	- hydraulic conductivity - grain size distribution - Atterberg Limits - triaxial tests - direct shear tests - moisture content - rock uniaxial compression test & intact rock modulus - point load strength test - Free swell - consolidation
Canal, trenches, pond areas Excavations and Cut Slopes	- slope stability - bottom heave - liquefaction - dewatering - lateral pressure - soil softening/ progressive failure - pore pressures	- subsurface profile (soil, ground water, rock) - shrink/swell properties - unit weights - hydraulic conductivity - time-rate consolidation parameters - shear strength of soil and rock (including discontinuities) - geologic mapping including orientation and characteristics of rock discontinuities	- hydraulic conductivity - grain size distribution - Atterberg Limits - triaxial tests - direct shear tests - moisture content - Dispersion/ Erodibility
Borrow area	Suitability for construction Volume of the borrow material (embankment materials)	Thickness of suitable layer Engineering property of the soil Shear strength of soil Shrink /swell properties of the soil	- hydraulic conductivity, grain size distribution, Atterberg Limits, triaxial tests, direct shear tests, moisture content, Erodibility, consolidation, compaction, chemical analysis, free swell
	Sand	Volume, Well grading or sorted,	- Specific gravity, Sieve analysis, Soundness,

Structure types	Engineering Evaluations	Required Information for Analyses	LaboratoryTesting
		Organic content	Mineralogical analysis
	Gravel	Durability Void Strength	- Specific gravity, Sieve analysis, Water absorption, Abration test, soundness, Mineralogical analysis
	Rock	Durability, Strength, Void, weathering	The tests as that of aggregate

2.6 ENGINEEERING GEOLOGICAL MAPPING DIVERSION PROJECT'S STRUCTURES SITE

An engineering geological map is a type of geological map which provides a generalized representation of all those components of a geological environment of significance in land use planning, in design, construction and maintenance as applied to civil engineering.

The purpose of engineering geology map is to provide basic information for planning, design, construction and maintenance of head work and associated structures. Such information is needed to assess the feasibility of the proposed engineering undertaking, and for the selection of the most appropriate type and method of construction, to ensure the stability of a structure in its natural setting, and to aid the performance of necessary maintenance.

An engineering geological map should fulfill the following requirements:

- It should describe the objective and information necessary to evaluate the engineering geological features involved in planning, in the selection of both a site and the most suitable method of construction.
- It should make possible to foresee the changes in the geological situation likely to be brought about by a proposed undertaking and to suggest any necessary preventive measures.
- It should present information in such a way that it is easily understood by professional users who may not be geologists.

Geological features represented on engineering geological maps of diversion and associated structures sites are:

- The characteristic of rocks and soils,
- Hydrogeological conditions
- Geomorphological conditions
- Geodynamic phenomena

Engineering geological maps should include interpretative cross-sections and an explanatory text and legend.

Engineering geological maps for characterization of diverssion/ micro dam project site:

- Need to contain information in regard to the influence of geological environment to the planned diversion and associated structures and vis versa.
- The maximum area to be covered on the map not more 100m towards upstream and down stream direction from the divesrsion weir axis and 400m towards upstream and down stream direction from micro dam axis.
- The scale of map for diversion site is not less than the produced topo map for the design of the structures of the projects

2.6.1 Description and classification of rocks and soils for engineering geological mapping

The boundaries of rock and soil units shown on engineering geological map of the proposed scale should delineate rock and soil units which are characterized by a certain degree of homogeneity in basic engineering geological properties.

The main challenge in engineering geological mapping is the selection of those geological features of rocks and soils which are closely related to physical properties, such as strength, deformability, durability, permeability, which are important in engineering geology. This is due to lack of regional data on the variability of engineering properties of rocks and soils or suitable methods and techniques been developed for determining them insufficient quantity, over large areas, quantitatively, quickly and cheaply. As the result, those geological properties which best indicate physical or engineering geological characteristics are usually used.

These engineering geological characteristics are:

(a) Mineralogical composition closely related to specific gravity, Atterberg limits and plasticity index;
(b) Textural and structural characteristics, such as particle size distribution, related to unit weight, porosity;
(c). Moisture content, saturation moisture content, consistency, degree of weathering and alteration, and jointing, related to the physical state of soils and rocks and indicating strength properties, deformation characteristics, permeability and durability.

Classification of rocks and soils on engineering geological maps should be based on the principle that the physical or engineering geological properties of a rock in its present state are dependent on the combined effects of mode of origin, subsequent digenetic, metamorphic and tectonichistory, and on weathering processes. This principle of classification make site possible not only to determine the reasons for the lithological and physical characteristic of soils and rocks, but also for their spatial distribution.

This is a basic principle of engineering geological mapping as of other geological mapping, based on not only the classification of individual rock samples but also the use of many individual rock samples, field observations and measurements to delineate uniform and continuous rock units.

2.6.1.1 Description of soil for engineering geological mapping

The sources of data for engineering geological mapping are mainly from the field description on top of information and data collected from the desk work. In order to classify and identify the boundaries of different engineering geological units of soil and rocks, systematic, scientific standardized description should be conducted during field survey.
Soil in civil Engineering project is defined to indicate fragmented material that can be excavated without blasting. In Engineering geology, an area is said to be covered by soil, i.e., to be mapped as a soil, the thickness should not be less than one meter.

Soil descriptions and units boundary delineations what we could obtain from the existing general geological maps and the one could be prepared by using areal photos and satellite images during the desk work is mainly generic (origin or source based). Descriptive soil names are usually supplemented to the types of deposit as alluvium, eluvium, residual colluvium, talus, scree, loess, aeolian etc. The variation in deposition, transportation and the sources of the soil influence its grain size texture which is one of the reasons for the variations in engineering properties of the soil.

Any loose material or soil can be categorized on the diversion project site engineering map in to four:

I. Residual soil
II. Transported soils
III. Pedogenic Materials, (fericrete, calcrete and silcrete)
IV. Organic Materials, (Organic accumulation like peat, musheg and swamp soils).

Soil in any area grouped in to different soil units on basis of field visual descriptions, laboratory tests results on the samples collected from the mapping area and also through combining laboratory results and field descriptions. Following the description, any soil group could be correlated with the description and group from the unified soil classification table (USCS).

Different field methods and terminologies to be used to characterize the engineering properties of soil for classification delimit on either undisturbed or disturbed samples are summarized from tables 2.1 to 2.6. Table 2.7 describes classes of soils which used internationally in both, civil engineering and the engineering geological environment.

Table 2-15: Description of soil material color

Lightness	Chroma
	Pinkish
Light	Reddish
Dark	Yellowish
	Brownish
	Greenish
	Bluish
	Grayish

Particle shape and composition may be described by reference to the general from of the particles, their angularity that indicates the degree of rounding at corners and edges, and their surface characteristics (see table 4.2)

Table 2-16: Terminology for describing shapes of soil particles

Forms of particle	Equi-dimensional, flat, elongated, irregular
Angularity	Angular, sub-angular, rounded, sub rounded
Surface characteristics	Rough, smooth

Table 2-17: Grades, description, field identification method and approximate strength of Soil by Manual Index

Grade **	Description	Field identifications	Approximate range of compressive strength (Mpa)
S1	Very soft clay	Easily penetrate several cm by fist	<0.025
S2	Soft clay	Easily penetrated several cm by thumb	0.025-0.05
S3	Firm clay	Can be penetrated several cm by thumb with moderate effort.	0.05-0.10
S4	Stiff clay	Readily indented by penetrated only with great effort	0.10-0.25
S5	Very stiff clay	Readily indented by thumb-nail	0.25-0.50
S6	Hard	Indented with difficulty by thumbnail	>0.50

**Grade S1 to S6 applies to cohesive soils for example, clays, silty clays and combinations of silts and clays with few sand, generally slow draining.

Table 2-18: Particle size range and their corresponding nomenclature

Size range	Particle	Size range	Particle
>256mm	Boulder	1/16-2mm	Sand
64-256mm	Cobble	1/256-1/16mm	Silt
4-64mm	Pebble	1/256mm	Clay
2-4mm	Gravel		

Table 2-19: some of frequently needed engineering properties of cohesive soils

Property	Silt	Clay
Specific gravity	2.63-2.67	2.55-2.76
Bulk density (Mg/m3)	1.80-2.16	1.48-2.16
Dry density (Mg/m3)	1.45-1.96	1.18-216
Void ratio	0.34-0.82	0.42-0.95
Liquid limit (%)	24-36	>25
Plastic limit (%)	14-25	>20
Permeability (m/s)	10^{-6} -10^{-9}	10^{-9} – 10^{-12}
Cohesion (KN/m2)	<70	15-200
Angle of friction coefficient	25-35	0
Consolidation (m2/yr)	12	5 to 12

Table 2-20: Some of frequently needed engineering properties of granular soils

Property	Gravels	Sands
Specific Gravity	2.5-2.8	2.6-2.7
Bulk density (Mg/m3)	1.44-2.3	1.4-2.2
Dry density (mg/m)	1.4 – 2.1	1.33-1.9
Angle of friction	33^0 - 45^0	27^0 – 46^0
Porosity (%)	25 – 40	25–50
Shear strength (Kpa)	180 – 550	100-400
Permeability (m/s)	10^{-1} – 10^{-6}	10^{-3} -10^{-6}

Table 2-21: Unified soil classification to be used for engineering geological mapping

Main divisions of soil group		Fines: Percentage finer than 0.06mm	Soil group	Group symbol	Sub group symbols	Soil Name
COARSE SOILS More than 35% of the material less than 60mm is larger than 0.06mm	GRAVEL Morethan 50% of coarse material is larger than 2mm	0 – 5	Gravel	G	GW GP GPU GPg	GRAVEL, well graded GRAVEL, poorly graded GRAVEL, uniformly graded GRAVEL, gap graded
		5 – 35	GRAVEL, silty Gravel with FINES GRAVEL, Clayey	GM GF GC	L, etc GCL, etc	GRAVEL, silty, of low plasticity GRAVEL, Clayey of low plasticity
	SAND More than 50% of coarse materials is smaller than 2mm	0 – 5 5 – 35	SAND SANDY, Silty SAND with fines SAND, Clayey		SW SP SPU SPg SML. Etc SCL etc	SAND, well graded SAND poorly graded SAND, uniformly poorly graded SAND gap graded SAND, Silty of low plasticity SAND, Clayey of

Main divisions of soil group		Fines: Percentage finer than 0.06mm	Soil group	Group symbol	Sub group symbols	Soil Name
						low plasticity
FINE SOILS More than 35% of the material less than 60mm and larger than 0.06mm	SILT AND CLAYES Gravelly or sandy	35 – 65	Silt gravel, fine soil, gravely clay, sandy	MG FG CG	MLG, etc CLG, etc	SILT, Gravely of low plasticity CLAY, Clayey of low plasticity
			Silt (m-soil) fine soil clay	MS FS CS	MSL, etc CLS, etc	SILT, Sandy of low plasticity CLAY, Sandy of low plasticity
		65- 100	SILT, sandy SILT fines, sandy CLAY, sandy	M F C	ML, etc CL CI CH CV CE	SILTY of low plasticity CLAY of low plasticity CLAY of Intermediate plasticity CLAY of High Plasticity CLAY of very high plasticity CLAY of extremely high plasticity
Organic Soil	Organic Sand, Silt or Clay			Description letter O is suffixed to any Symbol		
PEAT	Predominantly plant remains which may be fibrous, amorphous			Pt		
Materials coarser than 60mm is removed and recorded as cobbles (60mm-200mm) or boulders (over 200mm)						

** Properties of cohesive and non-cohesive soils and strength of cohesive soils by manual index shown on table 2.5 and 2.6 only helps to know or have an idea about range of properties. It is not advisable to take the parameters for design. It is always recommended to take the properties and parameters after field and laboratory tests for design purpose.

2.6.1.2 Description of rocks for engineering geological mapping

i. Origin based rock classification and description

Based on differences in origin and history of formation the rock types; however, differences in mechanical properties are very likely to be also present. Features such as the (in) homogeneity isotropy and (Dis) continuity are closely related to the origin and history of a rock type. This main genetical geological classification may thus be useful as a general engineering geological classification.

Igneous Rocks: originated by cooling of hot liquid molten rock

Sedimentary Rocks: Originated by transportation, chemical precipitation, etc. and deposits of weathered and decomposed material.
Metamorphic rocks: originated by action of intense heat and/or pressure on existing sediments or rocks. Planar metamorphic (gneiss, slate, schist) and massive metamorphic (marble, quartzite) can be distinguished

ii. Rock classification and description based on weathering

Weathering processes are subdivided as physical and chemical processes. Both process may be influenced by androgenic (internal) or exogamic (external) factors. Physical weathering or disintegration can be caused by:

- Periodical temperature

- Physical effects of plants Roots may split open existing rock separation planes.
- Internal rock stresses pressure
- Chemical weathering or decomposition can involve different chemical processes
- Hydration is a process of combination of rock minerals with water molecules
- Carbonation: the solution of rock minerals by water, which contains carbon dioxide.

Dislocation is the process of weathering of feldspar by carbonic acids, the results of which are clay minerals and solutions of silica and carbonates or bicarbonates of potassium, calcium or sodium.

Rocks can be classified in to five different glasses (groups) following their degree of weathering. Weathering degree (grade) and their standard description for using in the field mapping survey is presented below on the table 2.8.

Table 2-22: Rock weathering grade classification (folks, Dear Mass and Franklin, 1971)

Residual soil	The rock is discolored and is completely changed to a soiling which the original fabric of the rock is completely destroyed. There is a large volume change.
Completely weathered	The rock is discolored, discontinuities may be open and have discolored surfaces and the original fabric of the rock near the discontinuities is altered, alteration penetrates deeply in wards but core stones are still present.
Highly weathered	The rock is discolored, discontinuities may be open and have discolored surfaces and the original fabric of the rock near the discontinuities is altered, alteration penetrates deeply in wards but core stones are still present.
Moderately weathered	The rock is discolored, discontinuities may be open and surfaces will have greater discoloration with the alteration penetrating inwards, the intact rock is noticeably weaker, as determined in the field than the fresh rock.
Slightly weathered	The rock may be slightly discolored, particularly adjacent to discontinuities, which may be open and have slightly discolored surfaces; the intact rock is not noticeably weaker than the fresh rock.
Fresh rock	The parent rock shows no discoloration, loss of strength or any other effects due to weathering.

iii. Lithological classification and description of Rocks

Lithological classification of rocks has to contain generic name, associated micro or other geologic structure (if), dominant grain size, and composition. The description in terms these lithological terms are presented as follows.

Genetic group: (Igneous, Sedimentary (detrital, chemical/ organic), Metamorphic)

Geological Structures: (Bedded, Foliated, Massive)

Predominant grain size: (Very coarse-grained: Greater than 60mm, Coarse grained: 2 to 60mm, Medium grained: 0.06 to 2mm, Fine grained: 0.002 to 0.06mm, Very fine grained: less than 0.002mm, Glassy amorphous)

Composition (mineralogical): (Rock grains, Quartz, Feldspars and feldspathoids, Mafic (dark colored) and related minerals, Clay minerals, Carbonates, Salts: siliceous and carbonaceous materials, Glass).

Rock names are given to particular combinations of these features and correct naming requires recognition of the four attributes listed above; a simple, technically useful classification of fundamental rock types in each genetic group is given in table 2.9, 2.10 and 2.11.

The litho logical rock name is of primary importance because it indicates the genetic rock group and provides basic information on mineral composition and grain size, supplementary petro graphic properties may be used where necessary to quality the rock name, signifying for example a relative abundance of a particular mineral or indicating minor admixtures of other litho logical types. Guide to identification of igneous, sedimentary and metamorphic rocks after (BS 5930)

Description of rock material (sample)

Description of rock materials on geological mapping include:

1. Colour: rock color can evaluate using, rock color chart published by the geological Society of America (Anon. 1963).
2. Texture: Textural elements are used for description and classification. The most important is grain size, which for the predominant size of grain can be classified as presented above.

It is usually sufficient to estimate grain size by eye, which may be aided by a hand lens in the case of fine grained and amorphous rocks. The limit of unaided vision is approximately 0.06mm.
Many other aspects of rock texture may be used to amplify the description, such as:
Relative grain size: - for example uniform, non- uniform, prophyritic.

Grain shape:- may described by reference to the general form of the particles, their angularity which indicates the degree of rounding at edges and corners and their surface characteristics. See table 2.2, which is also applied for soil.

Fabric: - the spatial arrangement of grains in the rock may show a preferred orientation or lack of it, and may produce patterns by non-uniform arrangements of grains, crystals and groundmass.

Porosity: - the size, shape, orientation of pore or void spaces should be described.

Table 2-23: Classification of Igneous rocks depending on composition and grain size (source after BS 5930)

<table>
<tr><th>Pyroclastic</th><th colspan="5">Igneous</th><th colspan="3">Genetic Group</th></tr>
<tr><th colspan="6">Massive</th><th colspan="3">Usual Structure</th></tr>
<tr><td rowspan="2">At Least 50% Of Grains Are Of Igneous Rock</td><td colspan="2">Quartz, Feldspar, Mica, Dark Minerals</td><td colspan="2">Feldspars, Dark Minerals</td><td>Dark Mineral</td><td colspan="3">Composition</td></tr>
<tr><td>Acid</td><td>Intermediate</td><td colspan="2">Basics</td><td rowspan="7">Pyroxinite And Peridotite</td><td>Very Coarse Grained</td><td>60</td><td rowspan="7">Predominant Grain Size (Mm)</td></tr>
<tr><td>Rounded Grains , Agglomerate</td><td colspan="4">Pegmatite</td><td>Coarse Grained</td><td>2</td></tr>
<tr><td>Angular Grains Volcanic Breccia</td><td rowspan="2">Granite</td><td rowspan="2" colspan="2">Diorite</td><td>Medium Grained</td><td rowspan="2">Medium Grained</td><td rowspan="2">0.006</td></tr>
<tr><td>Tuff</td><td>Dolorite</td></tr>
<tr><td>Fine Grained TUFF</td><td rowspan="2">Rhyolite</td><td rowspan="2" colspan="2">Andesite</td><td rowspan="2">Basalt</td><td rowspan="2">Very Fine Grained</td><td rowspan="2">0.002</td></tr>
<tr><td>Very Fine –Grained TUFF</td></tr>
<tr><td></td><td colspan="4">Volcanic Glasses</td><td>Glassy Amorphous</td><td></td></tr>
</table>

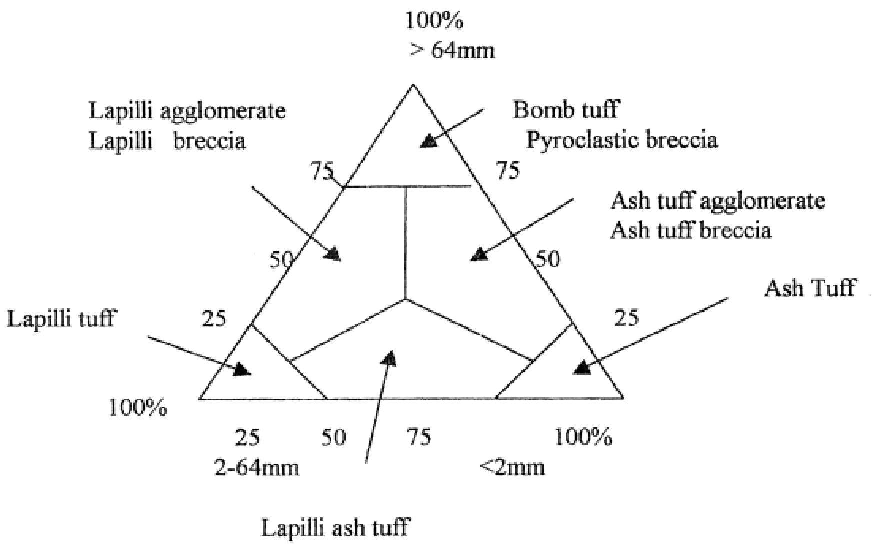
Table 2-24: Classification of Sedimentary rocks depending on composition and grain size (source after BS 5930)

<table>
<tr><th></th><th colspan="4">Detrital Sedimentary</th><th>Chemical Organic</th><th colspan="3">Genetic Group</th></tr>
<tr><th></th><th colspan="4">Bedded</th><th>Salt</th><th colspan="3">Usual Structure</th></tr>
<tr><td colspan="3">Grains Of Rock, Quartz, Feldspar And Clay Minerals</td><td colspan="2">At Least 50% Of Grains Are Carbonate</td><td>Salt Carbonate, Silica Carbonaceous</td><td colspan="3">Composition</td></tr>
<tr><td></td><td colspan="2">Grains Are Of Rock Fragments</td><td rowspan="7">Lime Stone (Undifferentiated)</td><td rowspan="3">Calcirudite</td><td rowspan="3">Salinc Rocks
Halite Anhydrite
Gypsum</td><td>Very Coarse Grained</td><td>60</td><td rowspan="6">Predominant Grain Size (Mm)</td></tr>
<tr><td rowspan="2">Rudaceous</td><td colspan="2" rowspan="2">Rounded Grains: Agglomerate
Angular Grains: Breccia</td><td>Coarse Grained</td><td>0.2</td></tr>
<tr><td>Medium Grained</td><td></td></tr>
<tr><td rowspan="2">Arenaceous</td><td colspan="2">Grains Are Mainly Mineral Fragments</td><td rowspan="2">Calcirudite</td><td rowspan="3">Calcarious Rocks
Limestone
Dolomite</td><td>Fine Grained</td><td>0.06</td></tr>
<tr><td colspan="2">Sand stone Fragments Are Mainly Minerals</td><td rowspan="2">Very Fine Grained</td><td rowspan="2">0.006</td></tr>
<tr><td rowspan="2">Argillaceous</td><td rowspan="2">Mudstone: Shale, Fissile Mudstone</td><td>Silt stone: 50% Fine Grained Particles</td><td>Calcirudite
Chalk</td></tr>
<tr><td>Clay stone: 50% Very Fine Grained Particles</td><td>Calcirudite</td><td>Silicious Chert Filint</td><td>Glassy Amorphous</td><td></td><td></td></tr>
</table>

Classification of pyroclastic rock

Ashes and their lithified equivalent stuffs have for a long time been subdivided in to three types: Vitric, Lithic and Crystal ashes and tuffs.

Classification based on particle size distribution. (Source: “Rocks and rock minerals” by Rv. Dietrich and B.J.Skinner, John Willey & Sons inc., 1979).



1. Ash Tuff:- composed of lithic, vitric or crystal, rounded or angular fragments of grain size distribution greater than 75% (by volume) less than 2mm in size well cemented, moderately strong when fresh.

2. Lapilli tuff:- composed of lithic, vitric or crystal rounded or angular fragments of grain size distribution greater than 75% (by volume) 2 to 64mm in size, well cemented, moderately strong to strong when fresh.
3. Bomb Tuff:-composed of rounded lithic fragments (bombs) of grain size distribution greater than 75% by volume greater than 64mm in size, well cemented, moderately strong to strong when fresh.
4. Pyroclastic Breccia:- composed of angular lithic fragments (blocks)of grain size distribution greeted than 75% by volume greater than 64mm in size, well cemented, moderately strong to strong when fresh.
5. Lapilli Ash Tuff:- Intermediate in composition between ash tuff and lapilli tuff, consisting of lappilli-sized fragments in matrix of ash tuff.
6. Lapilli Aggolomerate:- intermediate in composition between lapilli tuff and bomb tuff, consisting of rounded , litic fragments (bombs) greater than 64mm in size in matrix of rounded lithic lapilli fragments.
7. Lapilli Breccia:-intermediate in composition between lapilli tuff and pyroclastic breccias, consistingof angular lithic fragments (blocks) greater than 64mm in size in a matrix of angular lithic lapilli fragments.
8. Ash tuff Agglomerate:- intermediate in composition between lapilli tuff , consisting of rounded, litchi fragments (bombs) greater than 64mm in a matrix of ash and tuff.
9. Ash tuff Breccias:- intermediate in composition between ash tuff ,and pyoclasticbreccias, consisting of angular lithe fragments (blocks) greater than 64mm in a matrix size in a matrix of ash tuff.

Table 2-25: Classification of Sedimentary rocks depending on composition and grain size (source after BS 5930)

<table>
<tr><th colspan="2">Metamorphic Rock</th><th colspan="3">Genetic Group</th></tr>
<tr><td>Foliated</td><td>Massive</td><td colspan="3">Usual structure</td></tr>
<tr><td>Quartzite, Feldspar, Mica, Dark Minerals</td><td>Quartzite, Feldspar, Mica, Dark Minerals, Carbonates</td><td colspan="3">Composition</td></tr>
<tr><td colspan="2">Tectonic BRECCIAS</td><td>Very coarse grained</td><td rowspan="2">60
2
0.06
0.002</td><td rowspan="2">Predominant Grain Size (Mm)</td></tr>
<tr><td>Migmattite
Gneiss
Schist
Phyllite
Slate</td><td>Hornefels
Marble
Granulite
Quarzite
Amphibolite
Mylonite</td><td>coarse grained
Medium grained
fine grained
Very fine grained</td></tr>
</table>

iv. Geological structure based description of rocks

Rock separation planes

Rock is never completely be continuous; discontinuities of one of the following different types are always present:

a. Stratification planes: are present in sedimentary rock, due to irregularities of the sedimentation process.
b. Joints: are sets of parallel cracks in rocks.
c. Faults: are planes of discontinuity, along which relative movement has taken place.

d. Cleavage, schistosity are spatial orientations in the minerals of the rock due to a reorientation of these minerals under high temperature and pressure.

Horizontally layered bedding planes have no problem of instability. If permeable and impermeable beds are alternating, there could be seepage downstream. If beds are inclined there could be both instability and seepage problems.

Joints depending on type of filling, opening, dipping and roughness can have effect either on seepage or instability or both. Failure along a single discontinuity is a plane failure, while a wedge failure is caused by two discontinuities dipping towards each other.

When joints and fractures are dipping at a less angle especially downstream, there could be a high potential in seepage.

Rock Joint description

In engineering geological mapping rocky material description, the systems of Jointing, faulting etc. should be carefully described. (See Tables 2.14, 2.15, 2.16, 2.17, 2.18, 2.19)

In addition to the type and system of jointing, other aspects such as Spatial orientation, extent of joint, spacing of joint, opening, filling of joints, and roughness of the joints need to be described as follows:

A. Spatial Orientation

The orientation of a rock separation plane (Joint, faults regional or local) can be ensured with a compass. In engineering geology, usually the direction of dipping and the dipping angle are measured.

- The direction of dipping varies from 0-360 from North over east, south and west to North.
- The dipping angle varies from 0 for horizontal planes to 90 for vertical planes.
- The orientation measurements can be collected on a data sheets table.

For detail explanation of rose diagram and stereographic projection can be found in textbooks of any structural geology, for example Hobbs et al (1976, appendix I). Hoek and Bray (1977) and Hoek and Brown (1980) give excellent summaries of the stereographic projection technique for rock engineering purposes.

For illustrative purposes the rose diagram is often used. It is very suitable to plot traces of lineaments from satellite imagery or aerial photographs on this type of diagram. Also joint directions are often illustrated on rose diagrams. The direction –rose gives the statistical spread of the measurements of directions which fall in a given sector of the compass circle (say each 5 or 10 degrees and represent the result by drawing a line in the median direction of each sector, with a length proportional to the number of counts. However, only the direction or strike of discontinuity surfaces can be illustrated, not the amount of dip. (Figure 2.1)

The stereographic projection method is an excellent way of showing statistical spread of orientation data. The method is based on the representation of a spatial data on the surface of a sphere. Planes or lines going through the center of the sphere will cut the surface. In the case of a plane the intersection with the sphere will be a circle with the diameter of sphere (great circle). A line intersects the sphere in two points. Another way of representing plane in space is by normal to this plane going through the center of the sphere. This normal is called the pole of the plane. The stereographic projection equal angle projection and the equal area projection are projection from a sphere with a pattern of latitude and longitude lines on a two dimensional surface. Two types of projection are used, a polar projection and an equatorial projection.

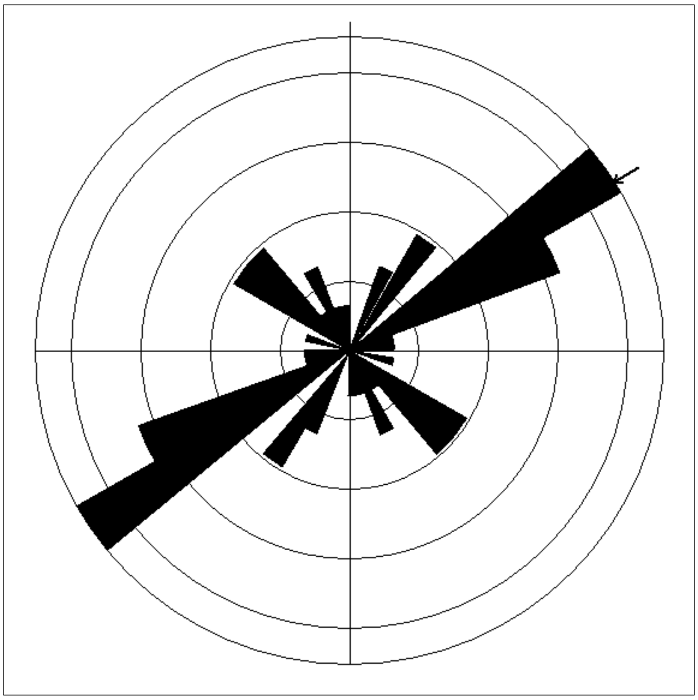


Figure 2-9: showing Joint Rose diagram of faults/fractures data (n = 82) of the Karadobi hydropower Site.

The diagram shows that there are three main sets of structures trending NE-SW (predominant), NW-SE and NNE-SSW in the area. Minor ENE-WSW to ESE-WNW and NNW-SSE are also present.

Table 2-26: Example of field description of rock joints

Rock type	Joint set (number)	Types of joint	Dipping direction (0-360)	Dip angle (0-90)	Extent (%)	Spacing (cm)	Opening (cm)	Filling	Joint roughness	Remarks
	A1	Cooling	165	90	100%	70	0-2	Clay	Plannar	-
	A2	Exfoliation	265	43	60%	50	1.5 -2	Silty	Undulating	-
	B1	Stratification	085	75	50%	35	3	Empty	Stepped rough	seepage
	B2	Tectonic joint	268	60	75%	200	2.5	Sandy	Planner smooth	-
	B3	Fault	009	55	95%	150	5-15	Sands-silty	Planner Rough	-
	C1	Schistocity	322	42	65%	25	2-3	Rusting		-
	C2	Techtonic joint	230	30	45%	45	5-10	empty		Water seepage

B. Extent

Many joints do not have strike completely. The percentage of the plane, which is developed as discontinuities, is called the extent of a joint. The extent of different joint sets in an exposure can have different extent percentage.

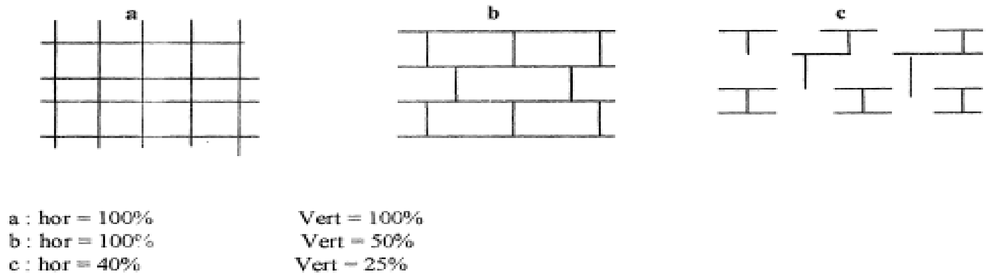


Figure 2-10: Illustration of extent of joint plane

Spacing

Spacing of joints refers to the average distances between the individual joints in a joint set measured in the direction perpendicular to the joint. It is expressed in mm, cm or m. the reverse value is the degree of jointing which is the number of joints per meter measured in direction perpendicular to the joints.

Spacing of discontinuities

Table 2-27: Description of joint spacing

Code (abbreviation)	Description	Spacing	(FI) ***
VC	Very Closely spaced jointed	Description	>20
C	Closely spaced jointed	50mm – 300mm	4-20
MC	Moderately closely jointed	300mm-1m	3-1
W	Wide jointing	1m -3m	
VW	Very widely jointed	Greater than 3m	

***Spacing of discontinuities is based on fracture index (FI) i.e., number of fracture per meter of core

C. Opening

The opening of a joint is the distance between the rock faces at both sides of the joint. The opening varies considerably at different locations along the joint; in that case it may be preferable to note maximum, minimum as well as average opening for the particular joint. The opening of joints can be of particular influence on the permeability and stability of a rock mass. For classification of opening or aperture see table 2.19.

D. Filling

The presence or absence of infilling material can have influence on the frictional, permeability etc. properties of joint; clay for instance can reduce the friction resistance considerably. The secondary permeability of rock mass also depends completely on the type of filling material. Clay filling can be so impermeable that they are barrier any water movements so that the secondary permeability is lower than the permeability. When joints are filled with coarse, crushed material or when joints are empty the secondary permeability is very high.

E. Roughness

The surface roughness of joints has a great influence on the frictional properties of the joints. Movement will not so easily take place along rough joints as along flat joints. Roughness scale can vary from large scale (waviness over several meters) to small scale (micro roughness over millimeters). In general terms used to describe roughness are, polished, smooth, rough, and very rough.

F. Planner

The surface planarity of joints has a great influence on the stability rock blocks along the joints. Movement will not so easily take place along plane joints as along flat joints. plannarity scale can vary from plane to stepped. In general terms used to describe planarity are: planner, curved, stepped and irregular

Recognition of fault

Faults are easier to map when they have distinct morphologic effects such as the displacement or abrupt ending of resistant key beds or sudden changes in their strike and dip or the formation of fault scarps, moist or seepage zones and lakes offsets or sudden changes in vegetation or tonal banding may further enhance such features.

Faults or fractures are zones of weakness with in the rocks, which are easily attacked by erosion, trenches or valleys forming along them. As a result fault or fracture controlled drainage will have straight and parallel or where two directions prevail angular forms.

Brecciated rocks in fault zones will often retain more water than the surrounding formations as a result the faults trace may show as a dark line because of the higher moisture content of the soil probably combined with denser vegetation. It may often be difficult to establish a character of a fault from aerial photographs alone even when its surface trace and perhaps also the direction of dip of the fault plane are clearly visible.

Morphological indications will be most clear under arid or semi-arid conditions, but also in humid regions with luxuriant vegetation the morphology my still offer considerable information. However, in heavily covered areas with thick vegetation and soil cover morphological indications may be blurred or absent. Under those conditions important information on the presence of faulting can be obtained from drainage.

Low angle faults frequently have less clear surface traces and continuous limitations may be absent. Especially low angle reverse faults and thrusts, which tend to have irregular and strongly curving traces, are difficult to interoperate from aerial photographs alone, even in well-exposed areas.

Major types of faults

- Normal or tension fault
- Thrust or reverse faults
- Wrenche or strike –slip faults

Joints or faults developed by failure in shear i.e, strain resulting from stress that causes parts of the rock to slide relative to each other in a direction parallel to their plane of contact. They tend to be clean-cut and tightly closed in un-weathered rock.

Fault breccia

During the process of faulting, the rocks are commonly broken up to greater or lesser extents. If the rocks on either side of the fault plane are hard, the fragments produced will be large and angular and are termed fault breccias. If the fault breccias are not cemented by secondary mineral it will have high permeability softer rocks and more intense movement involving the more resistance, may produce a quantity of fine, rock flour or fault gouge. Rock joint description is applied also for fault description. Joints, faults and shear zones are responsible for most of unsound rock encountering at dam, diversion and tunnel structures sites. Unless they are sealed, they may be permit leakage through foundations and abutments. Slight opening of joints on excavation leads to imperceptible rotations and sliding of rock blocks, large enough, however, to appreciably reduce the strength and stiffiness of rock the rock mass. Fault zones may be occupied by shuttered or crushed material and so represent zones of weakness, which may give rise toland sliding upon excavation for a dam. The occurrence of faults in a river is not unusual and this generally means that the material along the fault zone is highly altered. In such a situation a deep cut off will be necessary.

Table 2-28: Rock material Strength (ISRM, 1981); strength of rock by manual index

Grade	Description	Field identification	Approximate range of compressive strength (Mpa)
R0	Extremely weak rock	Indented by thumb nail	0.25-1
R1	Very weak rock	Crumbles under firm blows withpoint of geological hammer, can be peeled by pocket knife.	1-5
R2	Weak rock	Can be peeled by a pocket knife with deficulty. Shallow indentations made by firm blow with point of geological hammer	5-25
R3	Medium strong	Cannot be scrapped or peeled with a pocket knife specimen can be fractured with single firm blow of geological hammer	25-50
R4	Strong rock	Specimen requires more than one blow of geological hammer to fracture it	50-100
R5	Very strong rock	Specimen requires many blows of geological hammer to fracture it	100-200
R6	Extremely strong rock	Specimen can only be chipped with geological hammer	>200

Table 2-29: Joint apertures description (Indian Standard IS: 11315, (part 6), 1987

Opening	Nomenclature	
<0.25mm	Tight	Closed features
0.25-0.5	Partially open	
0.5 -2.5	Open	Gapped features
2.5-10mm	Moderately wide	
>10mm	Wide	
1-10cm	Very wide	Open features
10-100cm	Extremely wide	
>1m	Cavernous	

Table 2-30: Block shapes (IS : 11315, part 8, 1987

Massive	Few joints or very wide spacing
Blocky	Approximately equi dimensional
Tabular	One dimension considerably larger than the other two
Columnar	One dimension considerably larger than the other two
Irregular	Wide variations of block size and shape
Crushed	Heavily jointed to sugar cube

Table 2-31: Geologic nomenclatures of joints

Joint type	Characteristics
Longitudinal	Parallel bedding or foliation planes
Normal or cross	Intersecting bedding or foliation obliquely
Diagonal or oblique	Intersecting bedding or foliation obliquely
Foliation	Parallel foliations
Curvilinear	Forms parallel sheets or slabs, open curved

Table 2-32: Joint system, (Roy E. Hunt, Geotechnical Engineering Manual)

Joint set	Group of parallel joints
Joint system	Two or more sets or a group of joints with a characteristic pattern
Conjugate system	Two intersecting sets of continous joints
Orthogonal system	Three sets intersecting at acute angles to form wedges
Cubic system	Forming cubes
Rhombic	Three sets parallel but with unequal adjacent sides and oblique angles
Pyramidal	Sets intersecting at acute angle to form wedges
Columnar	Divides most in to columns three to eight sides, ideally hexagonal
Intense	Baddly crushed and broken rock without system , various shape and size of blocks

Table 2-33: Joint Persistence, (IS: 113115,(part3) 1987)

Persistence	Length (m)
Very low	<1
Low	1 – 3
Medium	3 – 10
High	10 – 20
Very high	>20

Rock quality Designation (RQD)

Deere et. Al. (1969) proposed a mass classification on the basis of Rock Quality Designation (RQD) determined by logging core;

Rock Quality Designation is a measure of the degree of brokenness of the rock in situ is determined by counting only the combined length of pieces of unweather core greater than or equal to 100mm or 4 inches in length as a percentage of the length of the coring interval.

This classification was set up with a view to use in tunneling and foundation engineering in which, clearly, the wider the joint spacing the greater the stability of the tunnel and the integrity of the foundation rock mass. However, if such a classification were applied to excavatability by digging machinery then an RQD of 90-100 would be described as very poor while that from 0-25 would be very good.

2.6.2 Importance for including of hydrogeological conditions

Hydrogeological conditions affect site selection, durability and even the safety of structures. Ground and surface water splay a prominent part in such geodynamic processes as weathering, slope movements, mechanical and chemical suffusion, the development of karstic conditions, volume changes by shrinking and swelling, and collapse in loessic soils. Rock and soil properties are often changed by groundwater. Groundwater may influence excavation and construction methods by flowing into excavations, by producing seepage forces and uplift pressures and by its corrosive action. Hydrogeological conditions may also affect underground waste disposal.

Natural groundwater and surface water regimes may be directly influenced by hydraulic structures and by extraction of groundwater, and indirectly by factors such as urbanization and deforestation which increase runoff, sediment load in streams and erosion, there by influencing other processes such as slope movement and sedimentation.

One aim of engineering geology, facilitated by the provision of hydrogeological data on maps, is the prediction of undesirable changes in the hydrogeological regime and the recommendation of procedures to avoid them. In engineering geological mapping, therefore, the following important information on hydrogeological conditions should be evaluated and represented on maps: the distribution of surface and subsurface water; infiltration conditions; water content; direction and velocity of groundwater flow; springs and seepages from individual water-bearing horizons; depth to water table and its range of fluctuation; regions of confined water and piezometric levels; hydro chemical properties such as pH, salinity, corrosiveness; and presence of bacterial or other pollutants.

On small-scale maps hydrogeological information is represented by symbols and numbers. On medium-scale maps the water table maybe represented by contours and its range of fluctuation indicated by numbers. In mountainous regions this is not possible and depth to water table and other features can only be shown by numbers. Both depths to confined water and piezometric levels can be shown by contours. On large-scale maps hydrogeological conditions are represented by isohypses, isobaths and isopiestic lines, with known fluctuations shown numerically.

2.6.3 Geomorphological conditions

Geomorphological mapping is helpful in explaining the recent history of development of the landscape such as the formation of valleys, terraces, slope configuration and the processes active in the landscape at the present time. It is an essential part of engineering geological mapping which can be carried out quickly and cheaply and is often a decisive factor in planning an engineering geological investigation.

Evaluation of geomorphological conditions in engineering geological mapping should be more than a simple description of surface topography. It should include an explanation of the relationship between surface conditions and the geological setting; the origin, development and age of individual geomorphological elements; the influence of geomorphological conditions on hydrology and geodynarnic processes. Also very important in engineering geology is the prediction of impending development of geomorphological features such as the lateral erosion of river banks, and collapse in karst or undermined areas.

Surface topography is shown by contours on maps of all scales. Point’s symbols are used to indicate significant geomorphological elements on small-scale maps. On medium and large-scale maps the actual boundaries and details of geo morphological features can be mapped.

2.6.4 Mapping of geodynamic phenomena

Geodynamic phenomena are those geological features of the environment resulting from geological processes active at the present time. Excluded are depositional or alteration processes as these are included in the description of rock and soils units. The geological features include those due to erosion and deposition, Aeolian processes, slope movements, permafrost, formation of karstic conditions, suffusion, and volume changes in soil, seismic and volcanic activity. All these features are important in engineering geological planning and construction. The amount of detail shown depends on the scale of the map. It is important to show not only the features but also the conditions favoring their development, their intensity and frequency of occurrence.

2.6.5 Zoning for engineering geological mapping

Comprehensive engineering geological maps present information in terms of engineering geological zoning. These are individual areas on the map which are approximately homogeneous in terms of engineering geological conditions and the area covered by any particular map sheet may be subdivided into a number of distinctive zoning units.

The detail and degree of homogeneity of each engineering geological zoned unit will depend on the scale and purpose of the map. On larger scale maps, zones are based on an evaluation of the uniformity of the structural arrangement and composition of rock and soil units, on hydrogeological conditions, and on geodynamic phenomena.

A map of special purpose engineering geological zoning would be prepared with a particular type of engineering undertaking in mind, for example, diversion projects, highways, dams, tunnels. On such a map the zoning units would be based on the analysis of geological phenomena and on geotechnical parameters, and, evaluated in terms of a particular engineering purpose.

2.7 GEOLOGY AND GEOTECHNICAL ENGINEERING REPORTS OF DIVERSION PROJECTS

2.7.1 Data presentation

Test boring logs and exploration test pit records can be prepared using soft ware capable of storing, manipulating, and presenting geotechnical data in simple one-dimensional profiles, or alternatively two-dimensional graphs (subsurface profiles), or three-dimensional representations.

Test location plans
A site location plan should be provided for reference on a regional or local scale.

The locations of all field tests, sampling, and exploratory studies should be shown clearly on a scaled plan map of the specific site under investigation. Preferably, the plan should be a topographic map with well-delineated elevation contours and aproperly-established benchmark.The direction of (magneticortrue) north should be shown.

If multiple types of exploratory methods are used, the legend on the site test location plan should clearly show the different types of soundings.

Subsurface profiles
Geotechnical reports are normally accompanied by the presentation of subsurface profiles developed from the field and laboratory test data. Longitudinal profiles are typically developed along the structure alignment, and a limited number of transverse profiles may be included for key locations such as at diversion structure weir foundations, main canal route. Such profiles provide an effective means of summarizing pertinent subsurface information and illustrating the relationship of the various investigation sites. The subsurface profiles, coupled with judgment and an understanding of the geologic setting, aid the geotechnical engineer in his/her interpretation of subsurface conditions between the investigation sites.

In developing a two-dimensional subsurface profile, the profile line (typically the weir axis, ponds, main canals or other structure foundation need to be defined on the base plan, and the relevant borings projected to this line. Judgment should be exercised in the selection of the borings since projection of the borings, even forshort distances, may result in misleading epresentation of the subsurface conditions in some situations.

The subsurface profile should be presented at a scale appropriate to the depth of the borings, frequency of the borings and soundings, and over all length of the cross-section. Generally, an exaggerated scale of 1(V):5(H) or1 (V):10(H) may be used.

The subsurface profile can be presented with reasonable accuracy and confidence at the locations of the borings. Generally, however, owners and designers would like the geotechnical engineer to present a continuous subsurface profile that shows an interpretation of the location, extent and nature of subsurface formations or deposits between borings. At a site where rock or soil profiles vary significantly between boring locations, the value of such presentations becomes questionable.The geotechnical engineer must be very cautious in presenting such data. Such presentations should include clear and simple warning explaining that the profiles as presented can not be fully relied upon. Should there be need to provide highly reliable continuous subsurface profiles, the geotechnical engineer should increase the frequency of borings and/or utilize geophysical methods to determine the continuity, or the lack of it, of subsurface conditions.

1.0 INTRODUCTION

- Back ground/ Overview previous works of the project
- Objective and Scope of the project
- Location and accessibility of the project (associated structure)
- Climate vegetation and current land use

2.0 REGIONAL AND SITE GEOLOGY

- Regional geology of the area
- Site geology
- Geomorphology
- Geologic structures of the area
- Hydrogeology
- Engineering geological map of the area showing locations of Test pits, trenches, geophysical survey lines and profiles through the infrastructure locations.

3.0 METHODOLOGY OF THE INVESTIGATION

- Planning
- Type of investigation methods and testing's
- Laboratory test types
- etc

4.0 FINDINGS

Diversion structure, Canal,etc and their sections as well as associated structural unit

- Review
- Site investigations
- Laboratory tests
- Analysis of both site investigation as well as laboratory tests in line with the infrastructure under considerations
- Construction material assessments (suitability, quantity, haulage distance, etc)
    a. Impervious materials (clay core, fill, canal lining, etc)
    b. Embankment (for canal fill)
    c. Rock sources (for production of aggregate, masonry work, rip-rap)

d. Sand as concrete making
e. water
- etc

5.0 FOUNDATION RECOMMENDATIONS

- Foundations bearing condition of the diversion structure site, pond or other structure
- Foundation depth and methods of excavation of the diversion structure /pond/ canal or other structure
- Foundation permeability conditions of dam site/ diversion structure weir/pond or other structure
- etc

6.0 CONCLUSIONS AND RECOMMENDATIONS

- Conclusions
- Recommendations

Appendixes:

Appendix I- Borehole/ test pits location Plan and subsurface Profiles
Appendix II- Test pits/Boring Logs and CoreLogs With Core Photographs
Appendix II- Field Insitu tests
Appendix IV-Geophysical Survey Data
Appendix V - Field PermeabilityTest data
Appendix VI – LaboratoryTest Results
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3 GEOTECHNICAL INVESTIGATION FOR MICRO DAM AND ASSOCIATED STRUCTURES PROJECT SITE

3.1 PLANNING

The goal in the initial planning stages of geological and geotechnical study planning is to develop an efficient investigation plan and to identify any geotechnical issues (previously known or new) that could potentially affect the projects scope, design or construction and performance of the project. To effectively manage the geotechnical factors affecting a project, the process must go beyond the collection data on the geotechnical conditions at the site.

It may be beneficial to conduct the field exploration in a phased sequence, consisting of a reconnaissance investigation and a preliminary subsurface investigation during the project definition phase and more detailed exploration conducted during the project design and construction phases.

If the subsurface exploration can be conducted in phases, it allows information obtained in the preliminary phase to be used in planning the exploration program for the detailed design phase. That is, the likely depths of additional test borings are known, the extent of the problem soil layers can be identified and studied in greater detail, the location of the proposed foundations, structures, and earthwork is better known, and the need for particular types of subsurface explorations or instrumentation can be identified.

The location of the site will play a part in the way the investigation is planned. For projects where mobilization costs for drilling equipment are high, the number of subsurface investigation phases should be minimized, even on fairly large projects. The studies and activities performed during the planning stage should be documented in Conceptual or Preliminary Level Geotechnical Reports.

It needs to become completely familiar with the proposed project elements in the preliminary office review. With an understanding of the project elements, the experts of Geotechnical evaluation team will prepare the subsurface exploration plan. The subsurface exploration program is a risk handling process; addressing the geotechnical parameters needed to design project elements and to fully explore the subsurface conditions within the project limits to minimize the potential for unforeseen conditions.

3.1.1 Desk Studies / office review of project site

Desk study and/ or review for Micro dam projects could be done in the same procedure as mentioned for that of the Diversion projects.

3.1.2 Short site visit

The field reconnaissance should be done with the preliminary plans in hand. Cross sections provided with the preliminary plans should be field checked. The cross sections are often generated by existing topographic data and may not accurately represent the existing ground surface, especially at areas with very steep changes in elevation or with dense vegetation.

Note the location, type and depth of any existing or abandoned structures or foundations and the location, size, and condition of any rock outcrops and arrange to have the structure surveyed using the original benchmark, if possible. Inspect for catchment and reservoir area, structure foundations and the stream banks up and down stream for evidence of sliding, scour and general stability.

Relate site conditions to proposed boring locations.

- Check access for exploration equipment and make an initial determination of what type of equipment might be best suited to the site conditions. If site preparation is necessary, note the type of equipment, such as a bulldozer, that may be needed for drilling equipment access.
- Note potential problems with utilities such as overhead and underground power, site access, private property or other obstructions. While utility will need to be obtained before the subsurface exploration begins, the locations will influence where explorations can be located.
- Note any water sources that could be used during drilling.
- Notes should be made as to which type of drilling is best suited to the site. Also note potential problems with borings such as shallow groundwater table, loose or heaving sands, cobbles and boulders, etc. Availability of water, if coring or mud rotary methods are anticipated, should be determined. Special sampling equipment needed, such as undisturbed sampling equipment, should be noted.
- Compare the topography of the site with that shown on maps and try to confirm the assumptions made during the office review concerning the site geology.
- Note the extent of any existing unstable slopes or erosion features.
- Photographs are valuable records of the site visit and should be labeled with the approximate stationing, direction of view, date, and a brief title.
- A record of the field visit should be kept and included in the project file. Measures should be taken to permanently archive any photographs taken. The record should list and describe significant site features as discussed above along with approximate stationing.

3.1.3 Development of the site investigation plan

3.1.3.1 Preparation of the site investigation plan

The site reconnaissance visit and office review results are generally used to develop the geotechnical designs required in the Design Phase.

The goal of the geotechnical investigation program is to obtain the engineering properties of the soil or rock and to define the lateral extent, elevation, and thickness of each identifiable soil/rock stratum, within a depth that could affect the design of the mocro dam.

The type, location, size and depth of the explorations and testing are dependent upon the nature and size of the project and on the degree of complexity and critical nature of the subsurface conditions.

In a laterally homogeneous area, excavating or advancing a large number of test pits may be redundant, since each sample tested would exhibit similar engineering properties. Furthermore, in areas where soil or rock conditions are known to be very favorable to the construction and performance of the foundation type likely to be used, fewer pittings may be justified.

3.1.3.2 Planning for test pits

Subsurface explorations for Micro dam may include test pits and associated insitu tests. These are simply manually or mechanically dug holes, often large enough for persons to work in, used to investigate subsurface strata, determine ground water conditions, or sample granular material sources. Small test pits may be used as percolation test pits or holes.

Planning for investigation using test pits excavation may include how to manage the following activities.

- General Sampling Requirements
- The need, use possibility of collecting undisturbed Sampling
- Selecting and conducting of relivant type of In-Situ Testings as per the site condition
- Evaluation of Groundwater Conditions in line with the influence on the micri dam structures

3.1.3.3 Planning for geophysical techniques

Geophysical techniques should be considered to enhance subsurface interpretation between test holes, to obtain information in areas where access is difficult for conventional test hole equipment, and to potentially reduce the cost of the geotechnical subsurface investigation program. Geophysical techniques are especially useful for defining lateral geologic stratigraphy, and can be useful to identify buried erosion channels, detailed rock surface location, over all rock quality, buried obstructions or cavities, etc., and, most recently (with improved electronics), shear wave profiles.

Geophysical testing should be used in combination with information from direct methods of exploration, such as SPT, DCP, etc. to establish stratification of the subsurface materials, the profile of the top of bedrock and bedrock quality, depth to groundwater, limits of types of soil deposits, the presence of voids, anomalous deposits, buried pipes, and depths of existing foundations.

There are a number of different geophysical in-situ tests that can be used for stratigraphic information and determination of engineering properties. Imaging method of Vertical Electrical Sounding (VES) and seismic refraction using a hammer are sufficient to characterize the foundations of Micro dams and associated structures.

3.1.3.4 Planning for subsurface exploration drilling

Geotechnical Engineer should submit the Exploration Plan to the Project Designer before explorations begin and alert the Project Designer of any unforeseen developments or deviations from the Exploration Plan.

The proposed locations of the subsurface explorations should have been checked for accessibility during the site reconnaissance (normally, the drilling supervisor will check for this). It may be necessary to shift the locations of some explorations due to local conditions, such as utilities, encountering obstacles such as boulders during drilling, or changes in engineering plans. The revised locations of these holes should be carefully plotted on the layout and the reason for the shift should be noted on the field log. Some tolerance in location of the explorations should be expected and communicated to the Drill Crew. The amount of tolerance will depend on the topography at the site, the expected soil conditions, stage of exploration, and type of structure.

The Drilling Inspector should be briefed as to what subsurface conditions to expect, any required methods of explorations, and the termination criteria for each subsurface exploration. They should contact the Geotechnical Engineer if any significant changes are encountered. It may be necessary to adjust the sampling intervals, the depth of explorations, or add explorations if the subsurface conditions are different than expected. If it becomes apparent that such changes will significantly impact the project budget or schedule, it is important to immediately contact the Project Designer to discuss the situation with them, and come to an agreement on the best course of action.

The Drill Crew should know how to get to the site, where to drill, what equipment to take, and what difficulties to expect. The Drill Crew's time should be spent in drilling and sampling and not in sending back for more equipment.

In preparing the Subsurface Exploration Plan, the Geotechnical Engineer identifies appropriate information such as:

- Type or method of explorations required.
- Sequence of drilling to allow for adjustment in the plan. For example, explorations in areas where soil conditions are unknown or problem soils are expected to be present should be performed in the first stages of the program, to allow for adjustment in sampling intervals or additional explorations to be added.
- Expected soil conditions. Attach field logs from nearby explorations, if available.
- Sampling intervals and types of samples to be obtained.
- Instrumentation and procedures for installation.
- Criteria for terminating each exploration – such as depth, refusal, and thickness of bearing layer, etc. If at all possible, the depth of all explorations should be estimated prior to doing the fieldwork. However, that is not always practical.
- Coordination of Drilling Inspector and Geotechnical Engineer regarding when and at what stages of the field exploration communication should take place.
- Equipment required and access needs
- Known permits required and regulations, and locations of all adjacent property lines
- Known utilities with locations and depths
- Coordination between the Geotechnical Engineer and Drill Crews is necessary to implement the field investigation program, to make sure that there are no logistical problems with the plan implementation.

3.2 GEOLOGICAL MAPPING

A map showing the distribution of rock units and geological structures across a given area, usually on a plane surface, is thus a geological map. Geological field mapping is the process of selecting an area of interest and identifying all the geological aspects of that area with the purpose of preparing a detailed geological report and a map to summarize the report.

A geological map will thus show the various rock types of the area, the structures, geological formations, age relationships, distribution of mineral ore deposits and fossils etc. and all these features may be super imposed over a topographic map or a base map. The amount of detail shown in a map depends largely on the scale and a smaller scale will naturally disclose finer detail. Basically, the quality of a geologic map will depend upon the accuracy and the precision of the field work. Further, still quality depends on the completeness with which certain data, both geologic and geographic are presented on the maps; and on the care with which scale, colors, conventions, etc are chosen to give the best results (Eckel, 1902). With the development of technology however, geological maps today are more precise than ever as a combination of accurate satellite imagery, aerial photographs, high tech geological equipment and Geographic Information Systems (GIS) advancements are applied.

3.2.1 Objectives of geological field mapping

Exploitation and planning of an area for certain propose requires the appreciation of basic geology and optimum utilization of a potential area requires that the rocks and soil units are mapped out. The reason why geological field mapping is carried is to create understanding of the spatial distribution and deformation of rock units at the surface and to use as a base map for developing a 3-dimensional understanding of the subsurface geology. In site exploration for the infrastructures of any micro dam project sites of any scale and associated structures sites, the geology of the area has to be thoroughly investigated and understood using a combination of geological method before a decision is made on the selection of that area. Geological mapping is usually the first task in any infrastructure foundations study as the question to what level the site is suitable to be answered using geological studies.

Guidance on detail procedures and steps are presented on the sub topic 2.2, of this guide line.

3.3 ENGINEERING GEOLOGICAL / GEOTECHNICAL SITE INVESTIGATION

3.3.1 General

For important structural projects programming and execution of site surveys is generally a growing process, by which site investigations develop from reconnaissance surveys to detailed fact-finding investigations (parameters of soil and rock properties). However, to arrive at an economic and effective site investigation program, the geotechnical engineer/foundation engineer should first collect all relevant data from the client and other sources, which is the most difficult part of foundation engineering. Much depends on the skill, feeling and experience of the foundation/geotechnical engineer, because the foundation design starts in fact at the very moment of the first discussions with the client, who is in most cases not familiar with soil/rock mechanics and the foundation/geotechnical engineer not (yet) familiar with the project.

Below are some aspects will be discussed, however these should not be regarded as a complete check-list, or a binding schedule to arrive at the necessary considerations for a good investigation program.

3.3.2 Objective of the Investigation

Objectives of the engineering geological and geotechnical investigation process are:

- Identifying and characterizing soils and rocks of the project site (surface & in subsurface)
- Extension and thickness of the soil and rock strata within the influence zone of the proposed project structure's foundation.
- Whether the soils and rocks under a project structure site can safely support the proposed project structures
- Groundwater conditions, considering the seasonal changes and the effects of extraction due to construction or development.
- Physical and engineering properties of the soil and rock formations, and effect of groundwater chemical contents on the coming structure.
- Impact of any planned excavation, grading or filling

- Hazardous conditions, including unstable slopes, active or potentially active faults, regional seismicity, floodplains, ground subsidence, collapse, and heave potential.
- Ground response to changing natural conditions and construction or development brought about by surface loadings from structures, un-loadings by surface or subsurface excavations, or un-loadings from the extraction of mineral resources.
- Availability of suitable construction materials for construction of the project within a reasonable distance in sufficient quantity.

3.3.3 Consideration and check list

Geological / geotechnical investigation activities vary from project to project based on type, components, phase and size of the project, and also the geology of the site. Consequently, planning the investigation through considering the necessary project and site conditions will help to obtain necessary result efficiently with in a proper time. The following basic conditions have to be considered planning geotechnical investigation and tastings for projects' site investigation. To get in to planning site investigation, information on the proposed project development should be gathered. Preliminary investigation should be carried out involving desk study, site inspection, image interpretation and reconnaissance site visit. The information which guides on methods of investigation planning include such as:

- Study phase of the project
- Project location and included components and dimension of the proposed project
- Design parameters of the projects required to be fixed.
- Previous works and Existing documents, if any

Project type and size

The other important input for planning the investigation program in order to set down precisely what the investigation should achieve for that particular project is type and size of project in consideration. In our case, **micro-dams, structures associated with for the Project (SSIPs).** The project specific geotechnical investigation objectives shall be defined in such a way that they can meet projects geotechnical parameters to be verified. Check lists of parameters to be verified for the mentioned SSIP project components are presented below.

Check list for Micro dam project site:

- Location: to suit topographic and geological situation
- Alternative sites, for comparison of costs and of geotechnical and other issues (This include two basic issues: selecting the best alteranative dam site from different options proposed the previous study phases or based on present additional data acquired during current)
- Depths to suitable foundation for: concrete dam, earth fill; core; filters; rock fill; plinth or grout cap;
- Nature of materials to be excavated, excavation methods, and possible uses of materials.
- Stability of excavation, support, and dewatering requirements.
- Permeability, compressibility and erodibility of foundations and foundation seepage controls
- Foundation treatment(s) required: grouting, drainage, slurry concrete, dental treatment, filter, blanket, others.
- Embankment zones, methods of placement, and control of quality, moisture and compaction.
- Stability of dam, and dam plus foundation in all situations.

- Monitoring systems: types and sitting.

Associated Spillway, intake structure and permanent outlet works

- Location and type , lay out.
- Excavation method(s), possible use for excavated materials.
- Stability of excavations, need for temporary/permanent support.
- Channel, need for lining/drainage.
- Need for protection of the discharge area, or for the excavation of a stilling basin.

Check list for Associated Reservoir site

- Water tightness
- Effect on regional groundwater-levels and quality & water sources
- Stability of slopes: inside and outside of reservoir rim
- Erodibility of soils-possibility of turbidity problems
- Siltation rates and likely location of deposits
- Checking for por potential borrow

Check list for construction materials

- Earth fill, for micro dam embankments
- Rockfill for bank erosion protections
- Concrete aggregates material(coarse and fine)
- Water sources
- Quality, quantity, proximity, ownerships, environmental

3.3.4 Methods of geological and geotechnical investigations

The methods of investigation depend on complexity of the area, objective of the investigation, size of the project, unfavorable geologic conditions, available related data previous work and the check lists of the parameter required for safe designing of the proposed structure on the site. After identifying all the necessary list of issues in line with the project under considerations, one could have clear views to plan methodologies and procedures to be included in a site investigation program, in order to address the issues which are presented in the check lists. Moreover, the sequence of investigation methods will also have an effect for improving the volume and quality of next investigations method. Accordingly, the most accepted sequence for the investigation method starts with Geological/engineering mapping then geophysical surveying, investigation through excavation of test pits, trenches and augers; and finally exploratory borehole drilling. Both Geological and geophysical investigation are called as Surface site investigation which implies characterization of grounding condition without any opening of the ground by methods such as geological / engineering geological mapping, geophysical survey (sometimes called indirect methods) and using different imageries.

3.3.4.1 Geological mapping

Issues of the site investigations to be addressed through engineering geological mapping are:

- Description of soil and rocks
- Identifying and characterizing geological structures
- Identifying and classifying land slide porn areas
- Classifying the rocks and soil units in to different geotechnical units based on the weathering.
- Collecting samples for laboratory tests
- etc

Guidance on detail procedures and steps are presented on the previous sub topic, geological and engineering geological mapping.

3.3.4.2 Geophysical survey methods

Geophysical method helps in determination of subsurface strata depth, extensions and also support information obtained by other methods. A number of geophysical methods are used in preliminary investigation of sub-soil strata. The methods can be used for the location of different strata and for a rapid evaluation of the sub-soil characteristics. However, these methods are very approximate. The geophysical methods frequently used in geotechnical investigation can be broadly divided into the two categories: Seismic methods and Electrical resistivity methods as described in the following sections. Selection is made based on nature of the engineering problem, available geological information, and site-specific criterion such as the depth of the target, required resolution, site accessibility, and cost. Manuals such as BS 5930, 1999, ASTM D6429, and ASTM D5753 could be referred to select suitable methods of geophysical investigation.

Seismic method

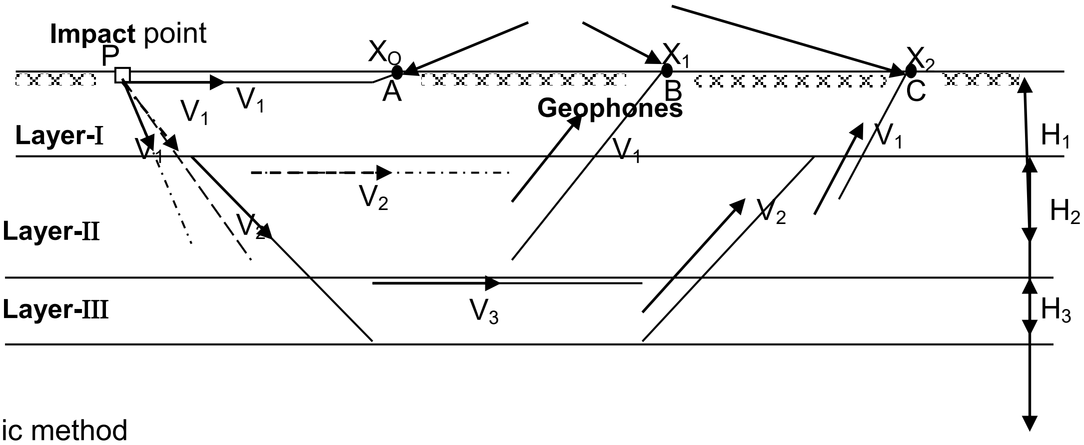
The seismic methods are based on the principle that the elastic shock waves have different velocities in different materials. At the interface of two different materials, the waves get partly reflected and partly refracted. Seismic methods of subsurface explorations generally utilize the refracted waves.

The shock wave is created by a hammer blow or by a small explosive charge at point P. The shock wave travels through the top layer of the soil (or rock) with a velocity V_1, depending upon the type of material in layer-I. The observation of the first arrival of the waves is recorded by geophones located at various points such as A, B, C. The geophones convert the ground vibration into electrical impulses and transmit them to a recording apparatus.

The basic equations of the refraction survey are derived based on the assumption that the velocity of the shock wave increases as the depth increases. In other words, it is assumed that $V_3 > V_2 > V_1$. At geophones located close to the point of impact, such as point A, the direct waves with velocity V_1 reach first.

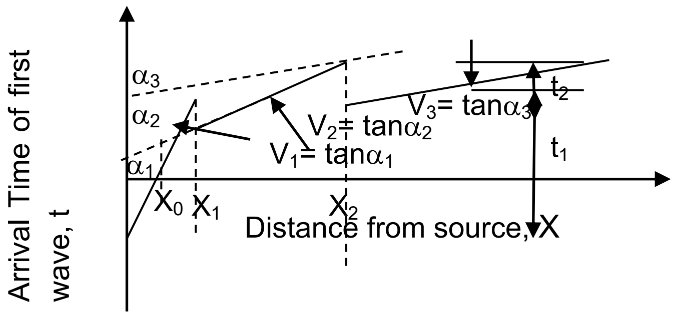
At points which are located away from the point of impact, such as point B, the refracted waves reach earlier than the direct waves. These waves start from point P, travel with velocity V_1 in the upper layer, get refracted at the interface, move with higher velocity V_2 in the second layer, emerge again at the interface and travel back to the ground at a lower velocity V_1 in the upper layer.

At points further away from the point of impact, such as point C, the waves which are refracted twice, once at the interface of the layers I and II, and once at the interface of the layers II and III, reach earlier.



Seismic method

For the determination of the thickness of different layers, a distance-time graph is plotted. The time (t) of arrival of the first impulse at various geophones is taken as ordinate and the distance (X) of the geophones from the point of impact P is taken as abscissa. Obviously, the velocity in any layer is equal to the reciprocal of the slope of the corresponding line. The slopes of the various lines are determined and the corresponding velocities computed.



Disatnce –time graph

Up to certain distance X_1, the direct waves in the layer I reach first. At this point, the first two lines intersect, which indicates that the direct wave travelling a distance X_1 with a velocity V_1 and the refracted wave travelling with a velocity V_1 in distance $2H_1$ and with a velocity V_2 in distance X_1 reach simultaneously, where H_1 is the thickness of the layer I. Thus

$$\frac{X_1}{V_1} = \frac{2H_1}{V_1} + \frac{X_1}{V_2}$$

Or $$H_1 = \left(\frac{V_2 - V_1}{V_2}\right)\frac{X_1}{2} \ldots\ldots\ldots\ldots\ldots\ldots \quad (3.1)$$

Eqn. 3.1 gives reliable results when the waves are produced by a sinusoidal force and not by impact. The following emphirical equation gives reliable results for impact shock.

$$H_1 = \frac{X_1}{2}\sqrt{\frac{V_2 - V_1}{V_2 + V_1}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3.2)$$

Likewise, the thickness of the second layer (H_2) is obtained from the distance X_2 corresponding to the point of intersection of the second and the third line in the drawing above. It is give by the relation

$$H_2 = 0.85H_1 + \frac{X_1}{2}\sqrt{\frac{V_3 - V_2}{V_3 + V_2}} \ldots\ldots\ldots\ldots \quad (3.3)$$

The procedure is continued if there are more than three layers

Draw backs of the seismic methods

1) The methods cannot be used if a hard layer with a greater seismic velocity overlies a softer layer with a smaller seismic velocity.
2) The methods cannot be used for the areas covered by concrete, asphalt, pavements or any other artificial hard crust, having a high seismic velocity.
3) If the area contains some underground features, such as buried conduits, irregularly dipping strata and irregular water table, the interpretation of the results becomes very difficult.
4) If the surface is layer is frozen, the method cannot be successfully used, as it corresponds to a case of harder overlying a softer layer.
5) The methods require sophisticated and costly equipment.
6) For proper interpretations of the seismic survey results, the services of an expert are required.

Despite above limitations, the method is extremely useful for the determination of the thickness of various strata and their characteristics. These surveys are useful for obtaining preliminary information the type and depths of various strata at a given site.

Electrical resistivity methods

The electrical resistivity method is based on the measurement and recording of changes in the mean resistivity or apparent specific resistance of the various soils. Each soil has its own resistivity depending upon water content, compaction, composition and many other factors. Rocks and dry soils have a greater resistivity than saturated clays.

To conduct the test, four electrodes, which are usually in the form of metal spikes, are driven into the ground along a stright line at equal disance. The two outer electrodes are known as current electrodes. The two inner electrodes are called potential electrodes. The mean resistivity of the strata is determined by applying a D.C. current to the outer electrodes and by measuring the voltage drop between the inner electrodes. A current of 50 to 100 milliamps is usually supplied.

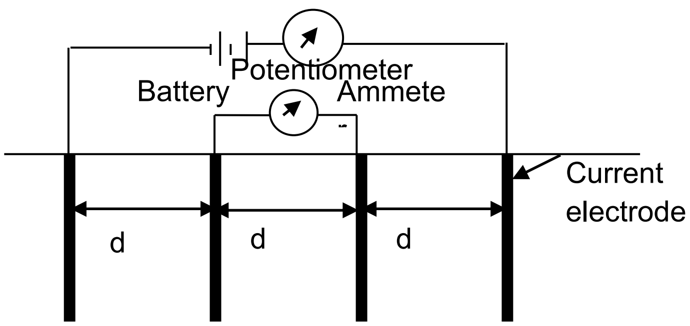
The mean resistivity (ρ) is give by the formula

$\rho = \frac{2\pi d V}{I}$.. 3.4

Where I = current suppled

d = spacing of electrodes

V = voltage drop



Resistivity method

Eqn.3.4 gives the mean resistivity up to a depth of d below the ground surface, as the depth of current penetration below the ground surface is approximately equal to the spacing of electrodes. The electrodes are moved as a group, and different profile lines are run across the area. The test is repeated after changing the spacing and again determining the mean resistivity up to the depth equal to the new spacing. The electrodes are moved as a group along different lines, as before. This method is known as *resistivity mapping*. The method is useful for establishing boundaries between different strata. The method is generally used for locating sand and gravel deposits within a fine-grained soil deposit.

For studying the vertical changes, the electrodes system is expanded, about a fixed central point, by increasing the spacing gradually from an initial small value to a distance roughly equal to the depth of exploration desired. The method is known as resistivity *sounding.* The method is useful in studying the changes in the strata with increasing depth at a point. The method can indicate sub-surface variation when a hard layer overlies a soft layer or vice-versa. It can also be used to locate the water table.

Drawback of electrical resistivity methods

1) The methods are capable of detecting only the strata having different electrical resistivity.
2) The results are considererabily influenced by surface irregularities, wetness of the strata and electrolyte concentration of the ground water.
3) As the resistivity of different strata at the interface changes gradually and not abruptly as assumed the interpretation becomes difficult.
4) The services of an expert in the field are needed.

Notwithstanding above limitations, the method is very rapid and economical for preliminary investigations.

3.3.5 Groundwater investigations

3.3.5.1 Purpose of the investigation

Groundwater investigations are of two types, those used to determine ground water levels and pressures and those used to determine the permeability of the subsurface materials. The former includes measurements to determine the elevation of groundwater surface or table and its variation with the season of the year, the location of perched water table, the location of aquifers, and the presence of artesian pressures and flow direction.

3.3.5.2 Observations of water levels and pressures

Existing wells - They are a good source of information of materials penetrated and the water level at the time of drilling. The owning organization of public or industrial wells may have records of water levels since installation and these are a source of additional information. In the case of private wells, records are not usually kept. The owner of the private well may also be able to give information as to the fluctuation of the water level and periods when the water has been low or the well has been dry.

3.3.5.3 Boreholes

It is common to establish the water table elevation at a site by measuring the depth to water in the boreholes. The length of time required for water levels in boreholes to stabilize at the groundwater level is a function of the permeability of the soil. There is no doubt that the water should be allowed to stand for a minimum period, preferably 24 hours, following completion of the hole. However, even under these circumstances, the reading can be obtained if readings are taken over a long period of time. In one shift a day drilling the groundwater level is usually observed as the first order of business in the morning. This gives 14-16 hours time for stabilization and this is frequently adequate.

Drilling mud obscures observation of the groundwater level owing to filter cake action and its specific gravity being greater than water. The depth to groundwater is usually measured by means of electric depth indicator consisting of a weighted probe attached to the lower end of a length of electrical cable that is marked at intervals to indicate the depth. When the probe reaches the water a circuit is completed and a meter mounted on the cable reel registers this. The electric indicator has the advantage that it may be used in extremely small holes.

3.3.5.4 Observation wells

The term "observation wells" is applied to any well or drilled hole used for the purpose of long-term studies of groundwater levels and pressures. Existing wells and boreholes in which casing is left in place are often used to observe groundwater levels. These, however, are not considered to be as satisfactory as wells constructed specifically for the purpose. The latter may consists of a standpipe or piezometer installed in a previously drilled exploratory hole or a hole drilled solely for use as an observation well. In cases where pressures in specific zones are required, bentonite or a similar material is used to seal the pizeometer with in the zone it is also customary to use a seal at the surface and to slope the fill at the top of the hole away from the pipe in order to prevent the entrance of surface water. The top of the pipe should also be capped to prevent the entrance of foreign material.

3.3.5.5 Effects of high groundwater table

Effect on hydraulic structures
Groundwater affects many elements of foundation design and construction. High groundwater table produces low bearing capacity, lateral pressure on retaining walls, construction hindrance, corrosive action, etc.

Therefore, groundwater table (GWT) shall be established as accurately as possible if it is within the probable construction zone. Some groundwater situations may have an important bearing on the choice of the type of dam to be constructed and on the estimates of costs of foundations.
Furthermore, groundwater investigation is required to identify the type of aquifer (unconfined, artesian and perched) and as an input data in the computation of permeability tests.
During borehole drilling or excavation it is important to determine:

- GWT and its variation with season of the year,
- The location of perched aquifer,
- The presence of artesian aquifer,
- Flow direction,
- Water loss during drilling due to cavities,
- Groundwater quality (chemical analysis),
- Pore pressure measurements, etc

Depending upon the type of the stratum, measurement of GWT shall be made after suitable time lapse of drilling operation. For example, for sand stratum, in 30 minutes after drilling; for silt and clay after 24hrs.

These time lapses are set depending upon the stabilization of GWT in the strata.

Commonly 12 to 24hrs, after drilling, is a practicable time for small and medium projects. An efficient method of measuring water table is by water level indicator of both visual and sonic type.

When the conditions so create that the crop root-zone gets deprived of proper aeration due to the presence of excessive moisture or water content, the tract is said to be waterlogged. To create such conditions it is not always necessary that groundwater table should enter the crop root-zone. Sometimes even if water table is below the root-zone depth the capillary water zone may extend in the root-zone depth and makes the air circulation impossible by filling the pores in the soil.
Water-logging renders the soil unproductive and infertile due to excessive moisture and creation of anaerobic conditions.

3.3.6 Test pit excavation

Detail procedures and guidance for test pit Excavation, descriptions insitu tests in the test pit already presented in the sub topic test pit excavation for Diversion project.

3.3.7 Investigation by exploratory core drilling

There are two basic types of rotary drilling: open hole (or full hole) drilling, where the drill bit cuts all the material within the diameter of the borehole; and core drilling, where an annular bit, fixed to the bottom of the outer rotating tube of a core barrel, cuts a core, which is recovered within the innermost tube of the core barrel assembly and brought to the surface for examination and testing. Rotary drilling for ground investigation is usually core drilling. When open hole drilling or coring, temporary casing is normally used to support unstable ground or to seal off fissures or voids, which cause excessive loss of drilling fluid. Drilling fluid additives or cement grouting may sometimes be satisfactory alternatives. The rotary drilling rig should be well maintained and should be capable both of controlling rotational speed and providing axial load and torque to suit the nature and hardness of the material penetrated, the diameter of the core barrel and drill string, drilling fluid and flushing system, weight of drill string and installation of temporary casing(s).

Figure 3-1: showing rotary core drilling at one of foundation Investigation site (drilling rig and accessories)

Drilling is in part an art, and its success is dependent upon good practice and the skill of the driller, particularly when coring partially cemented, fractured, weathered and weak rocks or superficial deposits. Good drillers have had adequate training, together with considerable experience

3.3.7.1 Drilling equipment

i. Rigs

Drilling rigs can be trailer mounted, truck mounted, skidding or chained. The depth of drilling vary from short to deep in hundred meters with manual or hydraulic feed depending on the capacity of the rigs. Drilling system can also classified as conventional and wire line. Wire line drilling system is good and preferable when required drilling depth is deeper than 50m. Rigs could be manufactured to drill Investigation holes vertically or inclined (mostly between 45 to 90 degrees).

ii. Down hole assemblies

Down hole assemblies are tools which are assembled together in order to conduct core drilling and retrieves cores for descriptions and laboratory testing. These include

- Core barrels (single, double, triple)
- Bits,
- Rods
- Casing and casing shoes
- Shelby (sampling tubes)
- Fishing tools (vary based on the size of drilling)
- In situ test assembly: SPT assembly (rod, cone, hammer), Packers (pneumatic/mechanically, double or single), deep meter

Size of core barrel and associated assembly vary depending on the diameter of holes (cores) to be drilled. Table 3.1 provides their sizes and respective symbols used in the borehole recordings.

Table 3-1: Symbols and abbreviations in borehole records (BS, 1981)

Borehole records	Core barrel designation	Core diameter (mm)	Hole (bits outer) diameter (mm)
Size in BS (British Standard)			
B	BWF,BWG or BNW	42	60
N	NFW, WFG or NWM	54.5	76
H	HFF or HNG	76	99
P	PWF	92	121
S	SWF	112.5	146
U	UWF	140	175
Z	ZWF	165	200
Miscellaneous size			
BX	BX(thin wall)	45	60
NX	NX(thin wall)	61	76
NQ3	NQ3(Triple tube wire line)	45	76
NQ	NQ (wire line)	47.5	76
HQ	HQ (wire line)	61	99
PQ	PQ (wire line)	83	121
NMLC	NMLC (triple tube)	52	76
HMLC	HMLC(triple tube)	63.5	99
Metric size			
T2 56	T2 56	42	56
TT56	TT56	45.5	56
T6 66	T6 66	47	66
T2 66	T2 66	52	66
T6 76	T6 76	57	76
T2 76	T2 76	62	76

Borehole records	Core barrel designation	Core diameter (mm)	Hole (bits outer) diameter (mm)
T6 86	T6 86	67	86
T2 86	T2 86	72	86
T6 101	T6 101	79	101
T2 101	T2 101	84	101
T6 116	T6 116	93	116
T6 131	T6 131	108	131
SK6 146	SK6 146	102	146

3.3.7.2 Describing or logging of recovered cores

Logging is recording of a continuous data of a borehole or shallow excavations. The records are presented in the form of graphic symbols, measured numerical values, test results, actual observations (water loss, core loss, gradation) and brief notes. The records can be further enriched from surface observations, natural and manmade cuts, outcrops and from any previous studies. As the core recovered will be soil or rock, the information expected to be extracted from soil is not totally the same as of that of the rocks.

i. Describing and logging of soil

Soil description/identification is the systematic, precise, and complete naming of individual soils in both written and spoken forms (ASTMD-2488,AASHTOM145), while soil classification is the grouping of the soil with similar engineering properties into a category based on index test results; e.g., group name and symbol (ASTMD-2487,AASHTOM145). It is important to distinguish between visual identification and classification to minimize conflicts between general visual evaluations of soil samples in the field verses a more precise laboratory evaluation supported by index tests. During progression of a drilling, the field personnel should only describe the soils encountered.

For Detail procedures and guidance of describing and logging of soil refer to the sub topic test pit excavation for diversion project.

ii. Rock core descriptions and logging

Rock descriptions should use technically correct geological terms, although local terms in common use may be acceptable if they help described istinctive characteristics. Rock cores should be logged when wet for consistency of color description and greater visibility of rock features. The guidelines presented in the "International Society for Rock Mechanics Commission on Standardization of Laboratory and Field Tests" (1978, 1981), should be reviewed for additional information regarding logging procedures for core drilling.

The rocks lithologic description should include as a minimum the following items:

Rock type

Rocks are classified according to origin into three major divisions: igneous, sedimentary, andmetamorphic. These three groups are subdivided into different types according to mineral and chemical composition, texture, and internal structure.

Color

Colors should be consistent with a Color Chart and recorded for both wet and dry conditions as appropriate.

Grain size and shape

The grain size description should be classified using the terms presented in table 2.4. Table 2-3 is used to further classify the shape of the grains.

Texture (stratification/foliation)

If there are significant non fracture structural features, it should be described.The thickness should be described using the terms in 3.2. The orientation of the bedding/foliation should be measured from the horizontal with a protractor.

Table 3-2: Terms to describe stratification

Description	Stratum thickness
VeryThicklybedded	>1m
Thickly bedded	0.5 to 1.0m
Thinlybedded	50mm to 500mm
VeryThinly bedded	10mm to 50mm
Laminated	2.5mm to 10mm
Thinly Laminated	<2.5 mm

Mineral composition

Mineral composition should be identified based on experience and the use of appropriate references.The most abundant mineral should be listed first, followed by minerals indecreasing order of abundance. For some common rock types, mineral composition neednot be specified (e.g. dolomite, limestone).

Weathering and alteration

Weathering as defined here is due to physical disintegration of the minerals in the rock by atmospheric processes while alteration is defined here as due to geothermal processes.Terms and abbreviations used to describe weathering or alteration are presented in table 4-8 of in the chapter 4, Engineering geological mapping.

Strength

Common qualitative assessment of strength while mapping or during primary logging of core at the rig site by using a geological hammer and pocket knife.The field estimates should be confirmed where appropriate by comparis on with selected laboratory tests.

Rock discontinuity

Discontinuity is the general term for any mechanical crack or fissure in a rock mass having zero or low tensile strength. It is the collective term for most types of joints, weak bedding planes, weak schistosity planes, weakness zones, and faults.

The spacing of discontinuities is the perpendicular distance between adjacent discontinuities.The spacing should be measured in centimeters or millimeters, perpendicular to the planes in the set.Table 6.5 presents guid lines to describe discontinuity spacing.

The discontinuities should be described as closed, open, or filled. Aperture is used to describe the perpendicular distance separating the adjacent rock walls of an open discontinuity in which the intervening space is air or waterfill ed.Width is used to describe the distance separating the adjacent rockwalls of filled discontinuities. The terms presented in Table 2-18 should be used to describe apertures.

Terms suchas"wide", "narrow" and"tight"are used to describe the width of discontinuities such as thickness of veins, faultgouge filling, or joints openings. Guidelines for use of such terms are presented on table 3.3 below.

Table 3-3: Guid line for describing and classifying discontinuity spacing

Opening	Description	Grouped
<0.1 mm	Verytight	Closed Features
0.1 - 0.25 mm	Tight	
0.25 - 0.5 mm	Partlyopen	
0.5 - 2.5 mm	Open	Gapped Features
2.5 - 10 mm	Moderatelyopen	
> 10 mm	Wide	
1-10 cm	Very wide	Open Features
10-100 cm	Extremely wide	
>1 m	Cavernous	

For the faults or shears that are not thick enough to be represented on the boring log, the measured thickness is recorded numerically in millimeters.

In addition to the above characterization, discontinuities are further characterized by the surface shape of the joint and the roughness of its surface.

Filling is the term for material separating the adjacent rock walls of discontinuities.Filling is characterized by its type, amount, width (i.e., perpendicular distance between adjacent rock walls) and strength. An example of field characterizing of joint filling is presented on table 4.12 in chapter 4. The strength of any filling material along discontinuity surfaces can be assessed by the guide lines for soil presented in the last three columns of Table 2-12 above. For non-cohesive fillings, then identify the filling qualitatively (e.g., fine sand).

Fracture Description

The location of each naturally occurring fracture and mechanical break is shown in the fracture column of the rock core log. The naturally occurring fractures are numbered and described using the terminology described above for discontinuities.

The naturally occurring fractures and mechanical breaks are sketched in the drawing column. Dip angles of fractures should be measured using aprotractor and marked on the log. For non-vertical borings, the angle should be measured and marked as if the boring was vertical. If the rock is broken in to many pieces less than 25mm long, the log mayb e cross hatched in that interval or the fracture maybe shown schematically. The number of naturally occurring fractures observed in

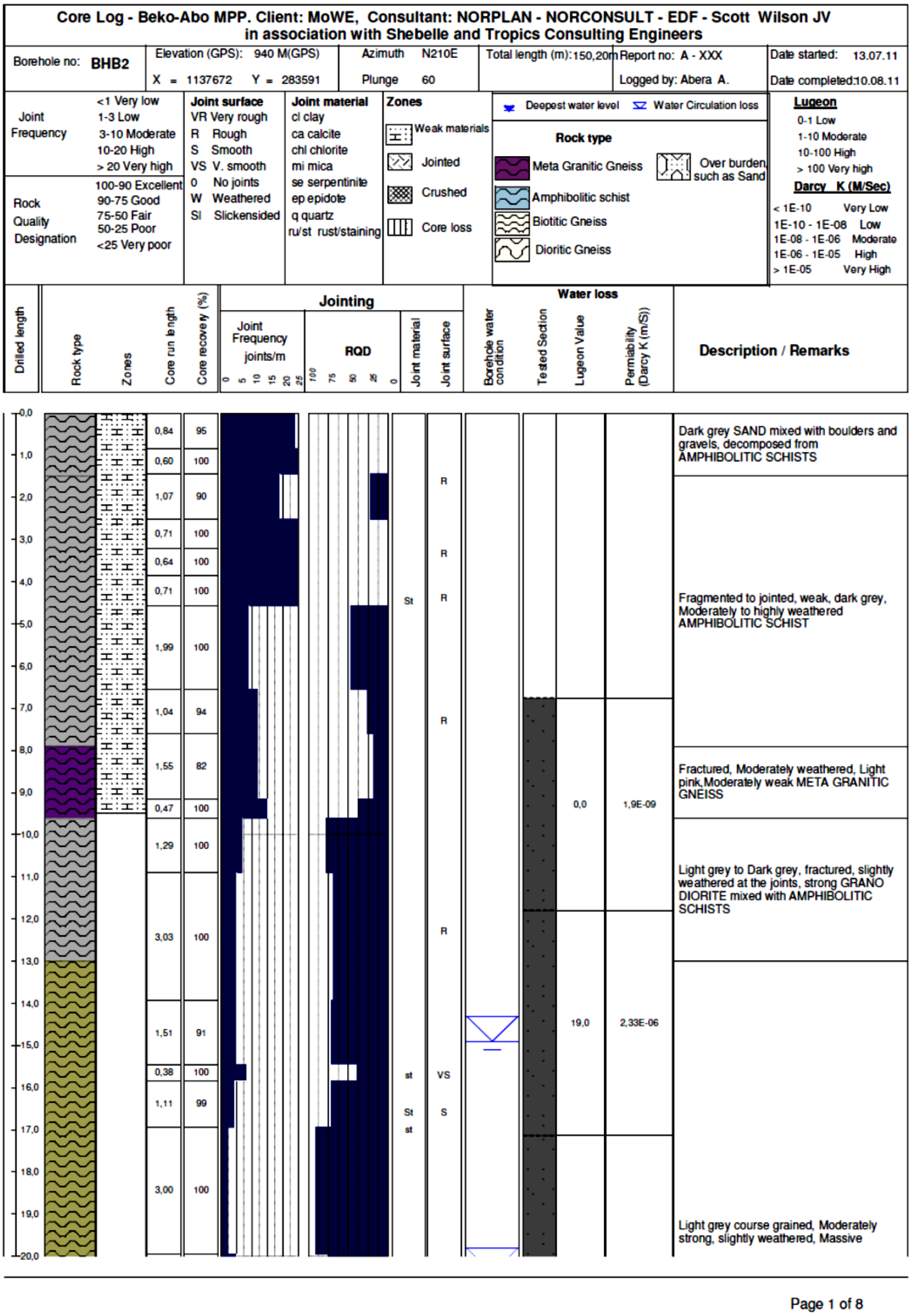


Figure 3-2: Typical log format

Each 0.5m of core should be recorded in the fracture frequency column. Mechanical breaks, thought to have occurred due to drilling, are not counted.

The results of core logging (frequencyand RQD) canbe strongly time dependent and moisture content dependent in the case of certain varieties of shales and mudstones having relatively weakly developed diagenetic bonds. Anot in frequent problem is"discing", in which an initially intact core separates into discs on incipient planes, the process becoming noticeable perhaps within minutes of core recovery.

The following criteria can be used to identifynatural breaks:

The various elements of the rock's description should be stated in the order listed above. For example: "Limestone, light gray, very fine-grained, thin-bedded, un weathered, strong"
The rock description should include identification of discontinuities and fractures.The description should include a drawing of the naturally occurring fractures and mechanical breaks.

Physical description
Other important aspects on the logging and description of rock cores are its physical descriptions. The term Physical description of rock cores include Total core recovery, solid core recovery and Rock quality description. Below are definitions, meanings and formuls to calculate these terms.

Total core recovery, TCR (%) = $\frac{lengthoftotalcorerecovered\ (dust,\frac{gravel}{intact}rock}{lengthofcorerun/advanced}X100$

Solid core recovery, SCR(%)= $\frac{lengthoftotalsolidcorerecovered\ (fulldiameters}{lengthofcorerun/advance}$X100

Rock quality designation, RQD (%)=$\frac{Sumoflengthsofintactpiecesofcoreswithlengths \geq 10cm}{lengthofcorerun\ (advanced)})X100$

TCR is an indication of compressibility of the stratum under the application of external loads from the structures. SCR and RQD are also a qualitative measure of the rock strength.The RQD is the most widely used rock parameter for indication of rock weakness/hardness for foundation, tunneling, shafts, underground and power house. At the same time it can indicate suitability of rock mass workability for rock fill, riprap and aggregate. Table 3-4 below shows quality comparison of RQD.

Table 3-4: Classification and description rock mass based on RQD

RQD	Description of rock quality	RQD	Description of rock quality
0-25%	Very poor	75-90%	Good
25-50%	Poor	90-100%	Excellent
50-75%	Fair		

In measuring and calculating RQD the actual drilling run length (3m, 1.5m or 0.5m) shall be used

3.3.8 In situ tests

Field in situ tests are often desirable where it is considered that the mass characteristics of the ground would differ appreciably from the material characteristics determined by laboratory testing. These differences normally arise from several factors, the most important of which are the extent to which the laboratory samples are representative of the mass, and the quality of sample that can be obtained for laboratory testing.

The in-situ conditions of stress, pore pressure and degree of saturation, and can be altered from an unknown in-situ stage by the sampling processes. Consequently, their influence cannot be accounted for in laboratory testing. The material tested in-situ by a field test is analogous to a laboratory sample, and can be considered as a field sample. The in-situ conditions of a field sample may be affected by the process of gaining access to the position, i.e. digging a trial pit, but usually the effect is very much less than for a laboratory sample. More obvious, however, are the controlling effects of the nature, orientation, persistence and spacing of discontinuities; the nature of any filling; and the size of sample required for it to be representative.

3.3.8.1 General

In situ tests requested during the evaluation of ground condition to fix the geotechnical parameters could be classified as strength and seepage characteristic determination tests. Both, strength and permeability of the sub surfaces are by different methods. The selection of suitable test type depends on formation type, availability of testing equipment, the project requirement. The following is the types of in-situ tests used frequently here, in Ethiopia.

3.3.8.2 In Situ strength tests

a. Standard Penetration Test (SPT)

Standard penetration test (SPT) is the most commonly used in-situ test, especially for cohesion less soils which cannot be easily sampled. SPT was originally developed in the late 1920s. The test is extremely useful for determining the relative density and the angle of internal friction of cohesion less soils. It can also be used to determine the unconfined compressive strength of soils.

Testing procedure

The test procedure was not standardized until 1958 when ASTM standard D1586 first appeared. It is essentially as follows:

- Drill a 60 to 200mm diameter exploratory boring to the depth of the first test
- Insert the SPT sampler (*also known as a split spoon sampler*) into the boring. The sampler is connected via steel rods to a 63.5kg hammer.
- Using either a rope and cathead arrangement or an automatic tripping machanism, raise the hammer a distance of 760mm and allow it to fall. This energy drives the sampler into the bottom of the boring. Repeat this process until the sampler has penetrated a distance of 450mm, recording the number of hammer blows required for each 150mm interval. Stop the test if more than fifty blows are required for any of the intervals, or if more than one hundred total blows are required. Either of these events is known as *refusal* and is so noted on the boring log.
- Compute the standard penetration number, N, value by summing the blow count for the last 300mm of penetration. The blow count for the first 150mm is retained for reference purposes, but not used to compute N because the bottom of the boring is likely to be disturbed by the drilling process and may be covered with loose soil that fell from the sides of the boring.
- Remove the SPT sampler; remove and save the soil sample.
- Drill the boring to the depth of the next test and repeat starting from steps **b** through **e** as required.

Thus, N value may be obtained at intervals no closer than 450mm. unfortunately; the procedure used in the field varies, partially due to changes in the standard, but primarily as a result of variations in the test procedure and poor workmanship. The test results are sensitive to these variations, so the N value is not as repeatable as we would like. The principal variants are as follows

- Method of drilling
- How well the bottom of the hole is cleaned before the test
- Presence or lack of drilling mud
- Diameter of the drill hole

- Location of the hammer (surface type or down-hole type)
- Type of hammer, especially whether it has a manual or automatic tripping mechanism
- Number of turns of the rope around the cathead
- Actual hammer drop height
- Mass of the anvil that the hammer strikes
- Friction in rope guides and pulleys
- Wear in the sampler drive shoe
- Straightness of the drill rods
- Presence or absence of liners inside sampler (this seemingly small detail can alter the test result by 10 to 30 percent)
- Rate at which the blows are applied

As a result of these variables, the test results will vary depending on the crew and equipment. Fortunately, automatic hammers are becoming more popular. They are much more consistent than hand-operated hammers, and thus the reliability of the test.

Although much has been said about the disadvantages of the SPT, it does have at least three important advantages over the other in-situ test methods: First, it obtains a sample of the soil being tested. This permits direct soil classification. Most of the other methods do not include sample recovery, so soil classification must be based on conventional sampling from nearby borings and on correlations between the test results and soil type. Second, it is very fast and inexpensive because it is performed in borings that would have been drilled anyway. Finally, nearly all drill rigs used for soil exploration are equipped to perform this test, whereas in-situ tests require specialized equipment that may not be readily available.

The standard penetration number has been correlated to soil characteristics such as: density, angle of shearing resistance,ϕ, unconfined compressive strength, as given in Tables 3.5 and Tables 3-6

Table 3-5: Correlation between number of blows (N), angle of internal friction and relative density of frictional soils (Terzaghi and peck)

N	0 – 4	4 -10	10-30	30 – 50	> 50
ϕ	$<28^0$	$28-30^0$	$30-36^0$	$35-40^0$	$>42^0$
Relative Density	Very loose	Loose	Medium	Dense	Very dense

Table 3-6: Correlation between number of blows (N), unconfined compressive strength and consistency of cohesive soils. (Terzaghi and Peck)

N	0 -2	2 - 4	4 – 8	8 -15	15-30	>30
$q_u(kN/m^2)$	0 -25	25 -50	50 -100	100 -200	200-400	>400
Consistency	Very soft	Soft	Medium	Stiff	Very stiff	Hard

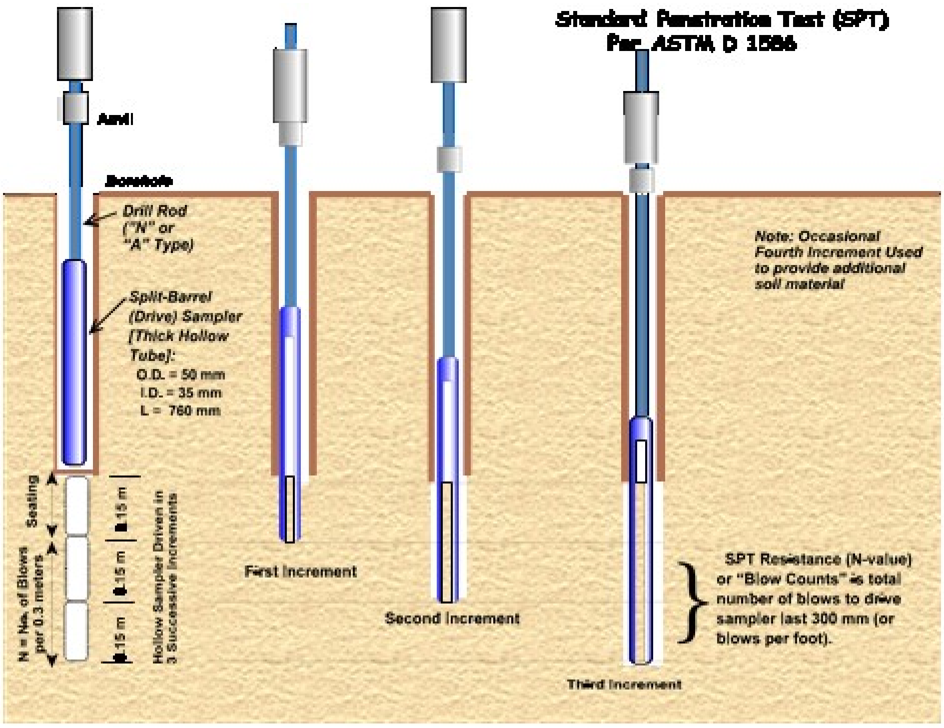


Figure 3-3: Standard penetration test (SPT) equipment and sequence of drivingsplit-barrelsampler duringthe standard penetrationtest.

The relationship between ϕ and D_r may be expressed approximately by the following equation (Meyerhof).

$\phi^o=30+0.15D_r$ …………………….. (3.5)

For granular soil, containing more than 5 percent fine sand and silt.

$\phi^o=30+0.15D_r$ …………………… (3.6)

For granular soil, containing less than 5 percent fine sand and silt. In the equations D_r is expressed in percent.

1. Corrections to test data

One can improve the raw SPT data by applying the following corrections

a. Correction for test procedure

The variations in testing procedures may be at least partially compensated by converting the measured N to N_{60} as follows (Skempton, 1986):

$$N_{60} = \frac{E_m C_B C_S C_R N}{0.60} \qquad (3.7)$$

Where

N_{60} = SPT N value corrected for field procedures

E_m = hammer efficiency

C_B = borehole dia

Meter correction

C_S = sampler correction

C_R = rod length correction

N = measured SPT N value

Many different hammer designers are in common use, none of which is 100% efficient. Most of the SPT-based design correlations were developed using hammers that had efficiency of about 60%, so Equation6.7 corrects the results from other hammers to that which would have been obtained if a 60% efficiency hammer was used.

b. Overburden correction

The SPT data also may be adjusted using an overburden correction that compensates for the effects of effective stress. Deep tests in a uniform soil deposit will have higher N values than shallow tests in the same soil, so the overburden correction adjusts the measured N values to what they would have been if the vertical stress , σ'_z was 100kPa. The corrected value, $(N_1)_{60}$, is (Liao and Whitman, 1985):

$$(N_1)_{60} = N_{60}\sqrt{\frac{100kPa}{\sigma'_z}} \qquad \text{..............................} \qquad (3.8)$$

Although Liao and Whitman did not place any limits on this correction, it is probably best to keep $(N_1)_{60} \leq 2N_{60}$ .This limit avoids excessively high $(N_1)_{60}$ values at shallow depths (Donald P.Coduto2001).

c. Dilatancy correction

Silty fine sands and fine sands below the water table develop pore water pressure which is not easily dissipated. The pore water pressure increases the resistance of the soil and hence the penetration number (N).

Terzaghi and peck (1967) recommend the following correction in the case of sily fine sands when the observed value of N exceeds 15

$N_c = 15 + ½ (N_R – 15)$ .... (3.9)

Where N_C= corrected value

N_R= recorded value

d. Permiability tests - Pressure (“Packer”) Test

A test in which water is forced under pressure into rock through the walls of a borehole provides a means of determining the apparent permeability of the rock, and yields information regarding its soundness. The information thus obtained is used primarily in seepage (permiability) studies. It is also frequently used as a qualitative measure of the grouting required for reducing the permeability of rock or strengthening it. Pressure tests should be performed only in holes that have been drilled with clear water.

The apparatus used for pressure tests in rock is illustrated schematically in Figure 6.4. It comprises a water pump, a manually-adjusted automatic pressure relief valve, pressure gages, a water meter, and a packer assembly. The packer assembly, shown in Figure 3.3 below, consists of a system of piping to which two expandable cylindrical rubber sleeves, called packers, are attached. The packers, which provide a means of sealing off a limited section of borehole for testing, should have a length at least five times the diameter of the hole. They may be of the pneumatically, hydraulically, or mechanically expandable type.

Pumping-in tests are conducted to determine the coefficient of permeability of an individual stratum through which a hole is drilled. These tests are more economical than the pumping-out tests. However, the pumping-out tests give more reliable values than that given by pumping-in tests. The pumping-in tests give the value of coefficient of permeability of stratum just close to the hole, whereas the pumping-out tests give the value for a large area around the hole.

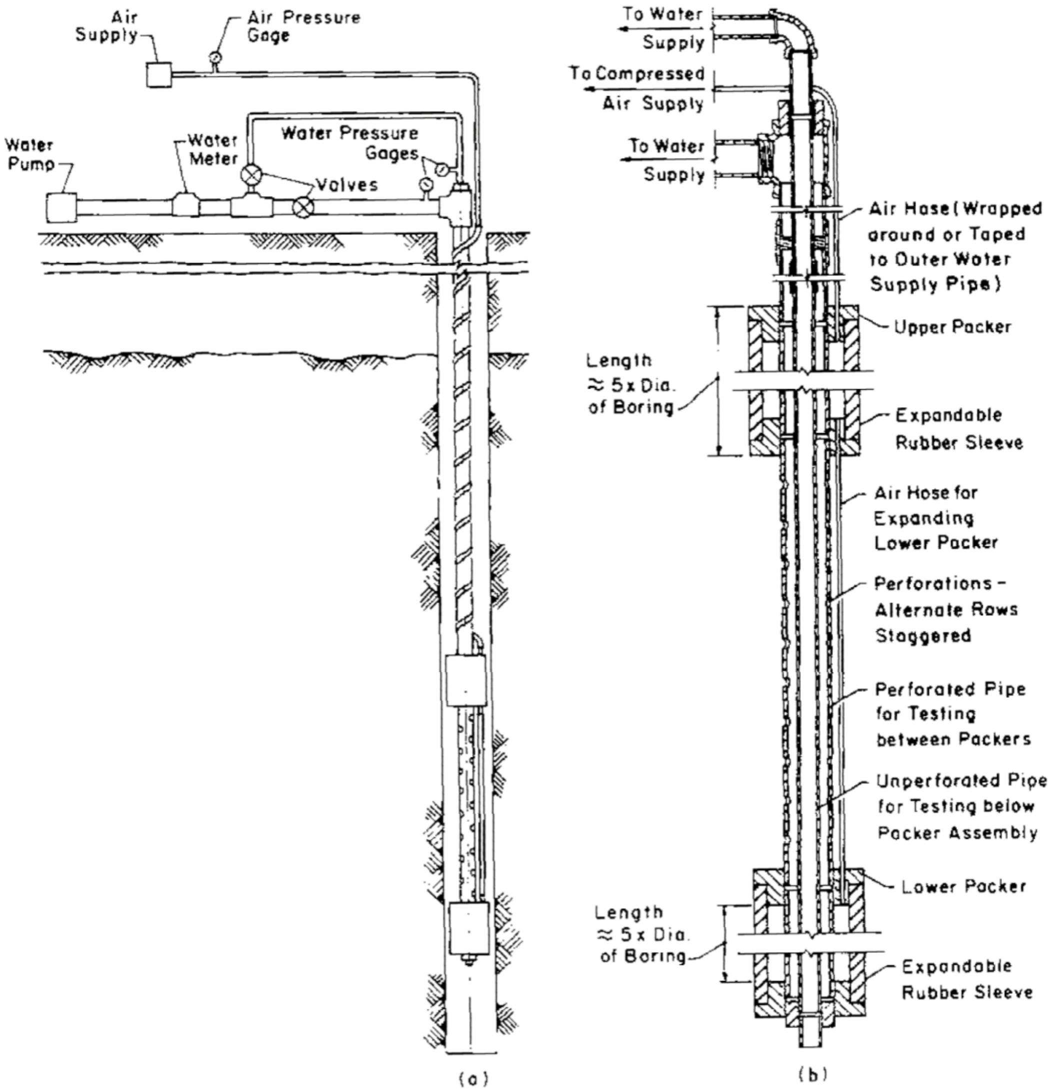


Figure 3-4: Packer-type pressure-test apparatus for determining the permeability of rock.

(a)Schematic Diagram ;(b)Detail of Packer Unit.(Lowe and Zaccheo,1991)

Pneumatic or hydraulic packers are preferred since they adapt to an oversized hole whereas mechanical packers may not. However, when pneumatic/hydraulic packers are used, the test apparatus must also include an air or water supply connected, through a pressure gage, to the packers by means of a high- pressure hose as shown in Figure 3.3. The piping of the packer assembly is designed to permit testing of either the portion of the hole between the packers or the portion below the lower packer. Flow to the section below the lower packer is through the interior pipe; flow to the section between the packers is provided by perforations in the outer pipe, which have an outlet area two or more times the cross- sectional area of the pipe. The packers are normally set 0.6, 1.5 or 3 m apart and it is common to provide flexibility in testing by having assemblies with different packer spacing available, thereby permitting the testing of different lengths of the hole. The wider spacing's are used for rock that is more uniform; the short spacing is used to test individual joints that may be the cause of high water loss in otherwise tight strata.

The test procedure used depends upon the condition of rock. In rock that is not subject to cave-in, the following method is in general use. After the borehole has been completed it is filled with clear water, surged, and washed out. The test apparatus is then inserted into the hole until the top packer is at the top of the rock. Both packers are then expanded and water under pressure is introduced into the hole, first between the packers and then below the lower packer. Observations of the elapsed time and the volume of water pumped at different pressures are recorded as detailed in the paragraph below. Upon completion of the test, the apparatus is lowered a distance equal to the space between the packers and the test is repeated. This procedure is continued until the entire length of the hole has been tested or until there is no measurable loss of water in the hole below the lower packer. If the rock in which the hole is being drilled is subject to cave-in, the pressure test is conducted after each advance of the hole for a length equal to the maximum permissible unsupported length of the hole or the distance between the packers, whichever is less. In this case, the test is limited, of course, to the zone between the packers.

The magnitudes of these test pressures are commonly 100, 200 and 300 kPa above the natural piezometric level. However, in no case should the excess pressure above the natural piezometric level be greater than 23 kPa per meter of soil and rock overburden above the upper packer. This limitation is imposed to insure against possible heaving and damage to the foundation. In general, each of the above pressures should be maintained for 10 minutes or until a uniform rate of flow is attained, whichever is longer. If a uniform rate of flow is not reached in a reasonable time, the engineer must use his/her discretion in terminating the test. The quantity of flow for each pressure should be recorded at1, 2 and 5 minutes and for each 5-minute interval thereafter. Upon completion of the tests at 100, 200 and 300 kPa the pressure should be reduced to 200 and 100 kPa, respectively, and the rate of flow and elapsed time should once more be recorded in a similar manner.

Observations of the water take with increasing and decreasing pressure permits evaluation of the nature of the openings in the rock. For example, a linear variation of flow with pressure indicates neither an opening that neither increases nor decreases in size. If the curve of flow versus pressure is concave upward it indicates that the openings are enlarging; if convex, the openings are becoming plugged. Detailed discussion for interpretation of pressure tests is presented by Cambefort (1964). Additional data required for each test are as follows:

- Depth of the hole at the time of each test,
- Depth to the bottom of the top packer,
- Depth to the top of the bottom packer,
- Depth to the water level in the borehole at frequent intervals (this is important since a rise in water level in the borehole may indicate leakage around the top packer. Leakage around the bottom packer would be indicated by water rising in the inner pipe).
- Elevation of the piezometric level,
- Length of the test section,
- Diameter of the hole,
- Length of the packer,
- Height of the pressure gage above the ground surface,
- Height of the water swivel above the ground surface, and
- A description of the material tested.

The formulas used to compute the permeability from pressure tests data are (from Earth Manual, US Bureau of Reclamation, 1960):

$K = \frac{\mathrm{Q}}{2\pi \mathrm{LH}} ln\left(\frac{L}{r}\right)$ For L $\geq 10r$... (3.10)

$K = \frac{Q}{2\pi LH} \sinh -1 \left(\frac{L}{2r}\right)$For 10r ≥ L ≥ r ……............................. (3.11)

where, k is the apparent permeability, Q is the constant rate of flow into the hole, L is the length of the test section, H is the differential head on the test section, and r is the radius of the borehole. The formulas provide only approximate values of k since they are based on several simplifying assumptions and do not take into account the flow of water from the test section back to the borehole. However, they give values of the correct magnitude and are suitable for practical purposes.

Packer test permeability values are also expressed in terms of lugeon unit. It has got a wide range of acceptance specially for grouting. One lugeon unit is equal to 1 litter of water taken per metre of test length per minute at 10 bar pressure.

Lugeon Value = litter of water/ meter/minute X $\frac{\Delta 10\text{bar}}{testpressureinbar}$ ……….. (3.12)

Equipment's required for packer test are

- Packers (double or single)
- Pipes or hose
- Pump
- Water meter
- Pressure gauge
- Deep meter for measuring water level
- Stop watch
- Others which includes water tanks, connections swivels etc

Table 3-7: Permeability classification of rocks (Lashkaripour and Ghafoori, 2002)

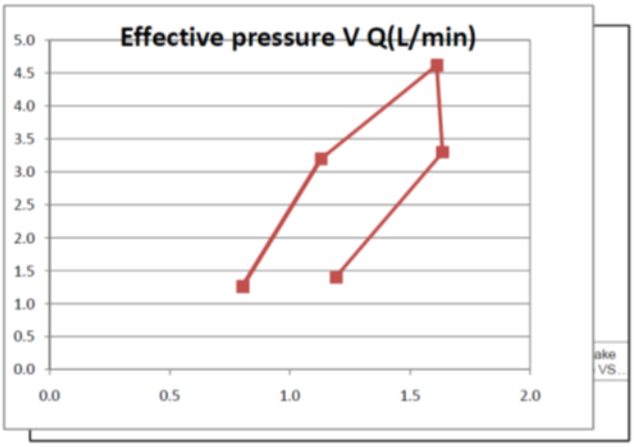
Lugeon Value	Permeability range/class
<3	Impervious
3 to 10	Low permeability
10 to 30	Medium permeability
30- 60	High permeability
>60	Very high permeability

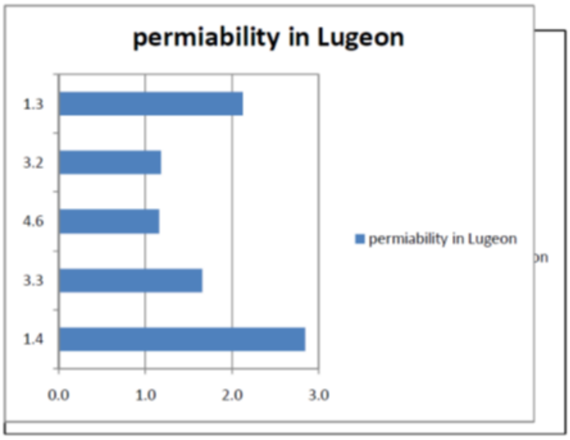
Table 3-8: Typical format and example of real water pressure test data collection and calculation

Permeability Determination - Water Pressure Testing

			Test	2
Borehole No	JVD- 05		Date	11/5/2013
Test section [m]	18.00	21.00	Weather	sunny
Borehole Depth [m]	30.00		BH diameter [mm]	91
Packer Position Depth [m]	18.00		Packer type	double
Water Level before test [m]	1.00		Packer diameter [mm]	54
Water level after test [m]	1.00		Hose length	40
Height of pressure gauge [m]	0.00		Time start	10.18
Height of water swivel [m]	0.00		Time finish	11.22
Length of Test section [m]	3.00		Inclination measured from horizontal	90

Time (min)	Step 1	Step 2	Step 3	Step 4	Step 5	Average
Gauge reading [bar]	1.30	3.20	4.52	3.10	1.16	
Effective Pressure mid section	1.4	3.3	4.6	3.2	1.3	
Water take (1t.)	11.92	16.36	16.12	11.3	8.02	
Flow rate (1t./min)	1.2	1.6	1.6	1.1	0.8	
L/min/m	0.4	0.5	0.5	0.4	0.3	
Lugeon	2.8	2	1	1	2.1	2
Coeff. Of Permeability k [m/s]	1.1E-07	6.2E-08	4.4E-08	4.4E-08	8.0E-08	6.8E-08





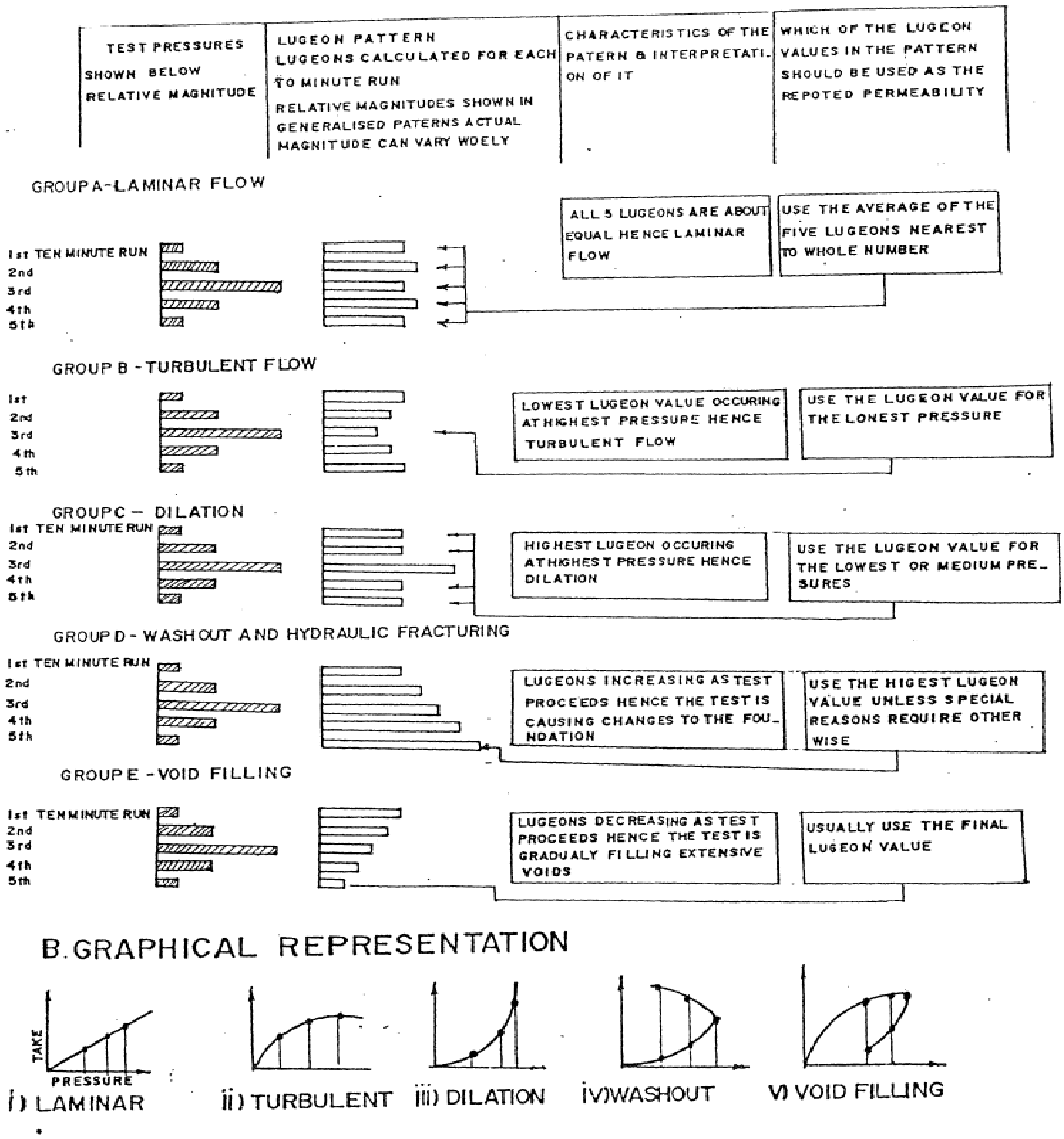


Figure 3-5: Selection of lugeon value by Huslsby (taken from study guide of geotechnical engineering by continental consultant & concert engineering)

3.3.9 Summary for Site Investigation Micro Dams and associated structures (SSIP)

3.3.9.1 General

Site investigation for geological and geotechnical investigation vary in type of the investigation as well as intensity or volume of work as per the structures planned to be constructed and the complexity of the site and the stage of study undertaken.

As discussed in the planning part of this manual, for a site investigation to be successful it has to be planned in orderly manner using appropriate field and laboratory equipment operated by experienced and skilled personnel.

Since the site complexity could not be standardized, as it varies from place to place, however considering the type of structures and stages of study, the following investigation types are foreseen. Summarized recommendations of site investigation are presented on the table 3.9 below.

3.3.9.2 Micro dam sites

At the initial stage any dam site selection is governed by the topography. Usually a narrowest section of valley is preferred as it needs less volume of construction materials.

Feasibility stage

Detail information of the surface and subsurface geological conditions of selected site has to be collected in this stage of study. The scale of engineering geological map is based on surveyed map used for design the project structures will satisfy this level of investigation. In this scale of map different geological and geotechnical units are identified and delineated which include the presence and types of geological structures. In addition to engineering geological mapping, subsurface investigation is also mandatory for this stage as estimation of cost and evaluation of benefits is one of the requirements for the stage.

The types of investigation to be conducted in these stages are:

- Test pits and trenches excavation associated tests
- Geophysical methods
- Core drilling and associated filed tests in the boreholes
- Laboratory tests

Spacing of drill hole is in the range between 150m to 250m depending on the complexity of site geological condition. Excavation of test pits, Auger holes and trenches between the holes is also foreseen in order to verify the depth of overburden. In most cases the location of the exploration holes, test pits and trenches are along the concluded micro dam axis. Moreover, at least one exploratory borehole has to be drilled at the upstream and downstream dam toes.

The depth of expletory borehole drilling is 2/3 to half of the hydraulic head (dam height) and/ or depth up to which weathering of bed rock or locate weak or depth undesirable layer below the foundation is clearly identified. If no competent or grout able rock is found in this range one or two holes may be drilled 3 to 5m in to bed rock.

Detail investigation stage

In this stage detail geological information has to be provided for all structures associated with dam in addition to the dam foundations. Common structures associated with micro dam are spillway, intake, power house sites and access roads.

The engineering geological map of this stage has to be prepared at scale between 1 :1,000 to 1 : 5,000 on topo map prepared by surveying with contour interval 1 to 5m. The mapping must comprise the sites of all associated structures and the quarries. In addition to the surface investigation, the map has to incorporate subsurface investigation (boreholes, test pits, geophysics, etc) and laboratory results.

Subsurface explorations for micro dams in this stage will depend on size of dam and type, extent and complexity of the fondation, which include depth of water table, engineering and index properties of the overburden, the depth to and type of bed rock, etc.

In this stage the investigation of all data required for detail design and preparation of construction drawings should be collected. Close work between geologists, geotechnical engineer and design engineer is essential to prepare an outline of the scope and extent of exploration.
Investigation of this stage comprises

- Additional explartary borhole drilling and associated insitu tests and excavation of test pits to characterize different types of foundation materials in relation to the designed features
- Emplying geophysical methods to define the foundation between the boreholes and the area as a whole. Use of approporiate geophysical methods based on the site condition is forseen.
- Permiability and water level of the foundation has to be defined through insitu tests and watre level measurments
- Groutability has to be checked by trial groutability test

The holes should be between 30m to 60m interval and the depth for the majority of the holes will be greater or equal to 1/3 of the dam height. More over, the foundation of the associated structures as well as the quarry site has to be evaluated through drilling short holes of test pits as per the site conditions.

3.3.9.3 Construction material site (Borrow)

Borrow site
Site investigation for borrow sites will be made either in feasibility or detail stages of the study. Based on the indication of the engineering geological map report, investigation for the assessments of borrow material suitability and sufficiency, excavation of test pits will be conducted 60mby 60m grid.

In case of rock fill dam for micro dams, the site investigations for the construction materials has to be done by drilling at least two short boreholes. The investigation has to be conducted in either feasibility or detail stages of investigations.

Table 3-9: proposed location, spacing, depths of exploratory boreholes for micro dam and associated hydraulic structure

Structure type	Location of the structure 's part	Investigation methods	Spacing boreholes (test pits) and features to be considered	Depth of investigation	remarks
Micro dam and associated structures	Dam axis	Test pitting, auguring and drilling with appropriate in situ tests & geophysical survey	Every 100m on left & right abutment, at the center and at the geological structures (if there exist anticipated structure)	Boreholes depth at least two third of the dam height	Engineering geological mapping of 1:5,000, geophysical survey has to be conducted before sub surface investigation
	Spill way	the same as above	2 BH &/or 2 TPs	1/2 of Dam H	
	U/S and D/S micro Dam toe	the same as above	One @ U/S & D/S	1/3 of Dam H	
	Intake structure	the same as above	2 Boreholes &/or TPS	10m	
	Reservoir area	the same as above except drilling	1:5000 mapping associated with test pit and augure excavations		
	Rock quarry	the same as above	2 BH and 2 TP	atleast 10m each	

Structure type	Location of the structure 's part	Investigation methods	Spacing boreholes (test pits) and features to be considered	Depth of investigation	remarks
Borrow areas		TP excavation and auguring	TP excavation is atleast every 70m by 70m grid	At least 3m below the existing surface	1:5,000 engineering geological mapping

3.4 CONSTRUCTION MATERIALS INVESTIGATIONS

3.4.1 General considerations

In geotechnical explorations, construction materials investigations comprise visual, field and labora tory assessments of locally available soil and rock materials. The assessments include impervious fills, embankment fills, transitions and drains, concrete ingredients, embankment protections, etc., of waterworks or other structures. Adequate coverage survey shall be carried out at the proposed site for identification of suitable sites for construction material. This shall cover:

- Investigation for identification of locations of potential quarries for construction materials like rock, aggregates, sand, soils and water; and proximity, quality, ownership of the land where the material is found, accessibility situation, environmental and social impact of quarrying and its mitigation measures as well as hauling distance including access route are to be examined;
- Estimation of quantity of material for each location, details of sample collection/testing of the materials, quality/suitability of the material, road maps showing the transport road up to the borrow area in relation to the construction site(s) shall be provided;
- Identifying the borrow areas; and preparation of location maps, road maps etc. showing the transport road up to the borrow area, relating the same to the construction site(s);
- Collection and evaluation of samples from borrow areas (rock, sand, soil, aggregate and water) for its suitability for different types of materials shall be collected for laboratory tests;
- The depth of the pits/auger holes shall depend upon the availability of the soils and economic exploitation. The borrow area shall be located at near the working site as possible. Pits/auger holes shall be taken in the proposed borrow area on 30 to 50 m grid depending on the uniformity of soil depth & type and representative samples collected/tested for different types of strata/soil to determine their properties and delineate the soil zones;
- For assessment of quantities, drill holes shall be taken especially for rock quarries;
- Required details of any other material as indicated in the earlier items shall be indicated.

3.4.2 Impervious materials

3.4.2.1 Properties required

Impervious materials are used in many parts of hydraulic structures such as in canals, ponds, etc. These impervious materials include clay soils, stabilized soils and concrete. In clay soils, there are desirable and non-desirable properties which the investigation has to look for use as impervious m aterial for blanket, canal lining, etc. These objectionable properties, after compaction, include:

- High shrinkage and swelling,
- Perviousness,
- Compressibility,
- Erodibility,
- Non workability as construction materials,

- Low shear strength, etc.

These non-desirable properties can also be reduced by additives, blending, high quantities, etc., but always the economics have to be there.

3.4.2.2 Clay fill and blanket

The impervious shall extend to full impervious depth or partially depending upon the extent of tolerable seepage allowed and other supplementary seepage prevention provided. Imperviousness, low shrinkage and swelling, good workability and low compressibility are highly needed.
As blanket, clays provide reduction of seepage loss, decrease in hydraulic gradient and uplifts to the structure.

Peat, organic clay and inorganic clays of high plasticity shall as much as possible be avoided for fill and blanket uses. Sources are from nearby excavations for appurtenance structures or borrow area of minimum distance from the structure.

3.4.2.3 Canal lining

On canal construction, linings are used to reduce water loss which in turn is required to conserve precious water, prevent water logging of adjacent lands and reduce the sizes of conveyance and headwork system.

Commonly used canal linings are clay, concrete, masonry, brick, etc. However, unless high degree of seepage control is required, clay lining is the most commonly used and locally available economic material.

For clay canal lining, high degree of impermeability, low shrinkage and swelling potential, erosion resistance and good workability are required. To achieve these properties thick linings, blending of good clays with silt, sand and gravel, and protective covers are needed. For this purpose, the sources are from canal excavation or nearby borrow area. Undesirable stones and plant growths shall be removed from the stockpile.

Before placing the lining materials on the bed and slopes of the canals, field and laboratory explorations and foundation preparations (excavation, scarifying and moistening) are required. Then blending (if necessary) and compaction in lifts at optimum moisture content are needed.

3.4.2.4 Canal fill

Depending upon the design requirements, earth embankments for main canals and laterals may consist of impervious or pervious soils placed loose, partially compacted by equipment or well compacted by rollers, or a combination of these. The selection criteria as earth embankment fill are not as such critical as that of linings, cores or blankets. Suitable material from canal excavation or borrow areas can be used. Usually cut and fill method with good compaction and lining is used.

3.4.3 Concrete aggregates

Aggregates are those chemically inert materials which when bonded with cement paste to form concrete. They constitute the bulk of the total volume of concrete and hence they influence the strength of concrete hydraulic structures to a great extent. The aggregates shall be hard, strong, dense, durable, and free from injurious amounts of clay, loam, vegetable and other foreign matters. Depe nding upon their size, the aggregates are classified as fine aggregate (sand) and coarse aggregat e (gravel).

3.4.3.1 Fine aggregate

Fine aggregates are those aggregate particles passing No 4 (4.75mm) sieve and retained on the No. 200(0.075mm) sieve. The term sand is used to represent this fine aggregate in a common sense.

Sand may be obtained from river, lake, sea and pits or produced through crushing rock. The sum of all types of deleterious materials in fine aggregate shall not exceed 5%. River sand usually suitable for concrete but mostly contaminated with mud and thus is advisable to wash such sand before use. Sea sand is also contaminated with salts from sea water which are liable to attack the steel reinforcement. Pit sand also generally suitable, but it is liable to contain silt or other organic matter. The other sand source is by crushing suitable hard rock to the required grading. There is a better control and qualities from this source. Therefore the economic comparison must be made between the naturally available sands in the locality and from crushing before deciding the source. The characteristic requirements of sand for hydraulic structure are more or less similar to other concrete works. Generally more exacting investigations are needed than for other purposes.

Angular sand has good interlocking property which results in a strong mortar while rounded grained sand does not afford sufficient interlock in the matrix. Soundness values of less than 8% in five reaction cycles are assumed to be satisfactory.

The fineness modulus of sand, which is the sum of the cumulative percentage retained on the num ber 4, 8, 16, 30, 50 &100 US standard sieves divided by 100, shall, for general concrete work, be maintained between 2.50 to 3.00, and the variation from the average on any job shall be held ± 0.1 if uniformity and close control of placement are desirable [Creager, et al, 1995].

The grading of sand can be divided into four zones. The grading becomes progressively finer from grading zone I (coarse sands) to grading zone IV (fine sands). Sands of grading zones I to III are mostly used for plain and reinforced concrete but IV is normally restricted for special mix of concret e (Table 3 .10).

3.4.3.2 Coarse aggregate

Course agregate is those aggregate particles predominantly retained on the sieve no 4 (greater than 4.775mm). Commonly the term Gravel is used to describe all the coarse aggregate. The sceintific and standards it is known as coarse aggregate. For sake of simplisity we use the commonly known term, that is gravel.

Gravel sizes are as described in Table 3.11. Gravels can be obtained from alluvial deposits or from crushed hard stones. The sum of the percentage of all type of deleterious substances in gravel shall not also, as sand, exceed 5%. As far as possible, flaky and elongated gravel shall be avoidedSoundness result (sodium sulphate test) shall be 10% for five cycles.

Gravels are usually obtained from crushed basalt, granite, gneiss and other suitable hard, dense and durable rocks.

In most cases of gravel, the material brought at site may be single sized aggregates (i.e.,ungraded) of nominal sizes 63 mm, 40mm, 20mm, l6mm, 12. 5mm and 10mm or it may be in the form of graded aggregates of nominal sizes 40mm, 20mm, I 6mm and 12. 5mm. The other is an all-in-aggregate (i.e., mixture of sand and gravel).

The size of the gravel used depends upon the nature of the work. The maximum size for mass concrete (as in dams) is 20mm and 63mm for plain concrete. For reinforced concrete, gravels having a nominal size of 20mm are generally considered satisfactory.

3.4.4 Rock sources

3.4.4.1 Properties required

Rocks for constructions may be required as massive rocks for diversion structure, stilling basins an d canals, drainage blankets and as masonry works in hydraulic structures. Rock sources shall sati sfy two main criteria: first the source shall able to produce rock fragments in suitable sizes, i.e., mo derate to slightly weathering and joints must exist for production and excavatability; secondly, the r ock material shall be hard, dense and durable to withstand destructive forces that encounter durin g placing or due to wave action or due to normal weathering.

Table 3-10: Grading limit for fine aggregates

Sieve size	Percentage passing			
	Grading zone I	Grading zone II	Grading zone III	Grading zone IV
10mm	100	100	100	100
4.75mm	90-100	90-100	90-100	95-100
2.36mm	60-95	75-100	85-100	95-100
1.18mm	35-70	55-90	75-100	90-100
600 micron	15-34	35-59	60-79	80-100
300 micron	5-20	8-30	12-40	15-50
150 micron	0-10	0-10	0-10	0-15

Table 3-11: Grading limit for gravel /coarse aggregates

Sieve size	Percentage passing for single sized gravels of nominal size of						Percentage passing for graded gravel of nominal size of			
	63mm	40mm	20mm	16mm	12.5mm	10mm	40mm	20mm	16mm	12.5mm
80mm	100						100	-	-	-
63mm	85-100	100					-	-	-	-
40mm	0-30	85-100	100				85-100	100	-	-
20mm	0-5	0-20	85-100	100			30-70	95-100	100	100
16mm	-	-	-	85-100	100		-	-	90-100	-
12.5mm	-	-	-	-	85-100	100	-	-	-	90-100
10mm	0-5	0-5	0-20	0-30	0-45	85-100	10-35	25-55	30-70	40-85
4.75mm	-	-	0-5	0-10	0-10	0-20	0-5	0-10	0-10	0-10
2.36mm	-	-	-	-	-	0-5	-	-	-	-

The common sources are bed-rock, river boulders, talus or surface deposits. Economic comparison must be made when different sources are available on a site. During field examination it is customary to describe the rock source by origin, colour, grain size and composition. Table 3.12 may aid in this regard.

a) Igneous rocks

Table 3-12: classification of rocks

Igneous	Acidic	Name	Composition	Usual structure
		GRANITE	Quartz, feldspar, micas, dark minerals	Coarse-grained (2-60mm)
		RHYOLITE		Fine-grained (0.002-0.06mm)
	Intermediate	PEGMATITE		Very coarse-grained (>60mm)
		DIORITE		Medium to coarse-grained(0.06-60mm)
		ANDESITE		Very fine to fine-grained (<0.06mm)
		OBSIDIAN		Glassy amorphous
	Basic	GABRO	Feldspar, dark minerals	Coarse-grained (2-60mm)
		DOLERITE		Medium-grained (0.06--2mm)
		BASALT		Very fine to fine-grained (<0.06mm)
	Ultrabasic	PYROXENITE PERIDOTITE	Dark minerals	Coarse-grained (2-60mm)
Pyroclastics	Rounded grains AGGLOMERATE		At least 50% of grains are of igneous	Very coarse-grained (>60rnrn)
	Angular grains VOLCANIC BRECCIA			Coarse-grained (2-60mm)
	TUFF			Medium-grained (0.06-2mm)
	Fine-grained TUFF			Fine grained (0.002-0.06mm)
	Very fine-grained TUFF			Very fine-grained (<0.002mm)

b) Metamorphic rocks

Foliated	Name	Composition	Usual structure
	MIGMATITE	Quartz, feldspar, micas, dark minerals	Coarse-grained (2-60mm)
	GNEISS		Medium to coarse-grained(O .06-60mm)
	SCHIST		Fine to medium-grained(0.0020.06mm)
	PHYLLITE		Fine-grained (0.002-0.06mm)
	SLATE		Fine-grained (0.002-0.06mm)
	Tectonic breccia	From both the foliated and non-foliated minerals	Very coarse-grained (>60mm)
	Amphibolite		Fine to medium-grained (0.002-2mm)
	Mylonite		Very fine-grained (<0.002mm)
Non foliated	HORNFELS	Quartz, feldspar, micas, dark minerals, carbonates	Medium to coarse-grained(0.06-60mm)
	MARBLE		Medium-grained (0.06-2mm)
	GRANULITE		Medium-grained (0.06-2mm)
	QUARTZITE		Medium-grained (0.06-2mm)

c) Sedimentary rocks

Clastic	Name	Composition	Usual structure
	CONGLOMORATE	Any kind of rock or mineral	Rounded pebbles: >2mm
	BRECCIA		Angular pebbles: >2mm
	SANDSTONE	Quartz or feldspar and quartz	Sand size grains: 0.0625-2mm
	SILTSTONE	Mostly quartz and some clay	Very fine grains: 0.004-0.06mm
	SHALE	Mostly clay and mud, some mica	Microscopic grains and flakes
Nonclastic	LIMESTONE	Calcite or shells and fragments	Coarse to microscopic
	CHERT(FLINT)	Chalcedony (precipitate)	Microscopic
	ALABASTER (ROCK GYPSUM)	Gypsum or anhydrite (Evaporite)	Coarse to microscopic
	ROCK SALT	Halite (evaporite)	Cubic crystals
	PEAT, COAL	Plant fragments, carbon compounds	Coarse to microscopic

3.4.4.2 Stone masonry work

Rocks for masonry purposes are required in different parts of hydraulic structures. These are masonry walls for camp housing, different thick walls in diversion structures weir, canal falls, canal stone linings, retaining walls, etc.

In addition to strength requirements of the stones, shapeability and dressebility of the rock fragments are necessary. Commonly ignimbrite (except soft tuft) and trachytic rocks are preferable but granite, basalt, etc, can also be used.

3.4.4.3 Water

Large amount of water is usually required in every construction site. Water is required for concrete mixing, curing, compaction for embankments, foundation moistening before placement, etc. Water for the above purposes can be obtained from river, spring, sea, lake, pond, groundwater, rain, etc.

Water to be used in masonry and concrete works shall have the following properties:

- It shall be free from injurious amounts of oils, acids, alkalies, organic and inorganic impurities and
- It shall be free from mud, iron, vegetable matter or any substance which is likely to have adverse effect on concrete, masonry or reinforcement.

pH values between 6 & 9 are usually don't need special precautions, but out of these ranges speci al protective measures such as increasing the cement proportion in the mix, increasing the dimens ions of the section or corrosive resisting cement types shall be used. Similarly, sulphate attack (S0 3) is small if the water contains a concentration of sulphate less than 300mg/l, otherwise similar re medial measures have to be taken on the concrete works.

Some of the tests required in water for construction purposes include impurities and suspended m aterial determination and chemical tests such as pH, sulphate, chloride, etc. mention BS standard

3.5 LABORATORY TESTS AND INTERPETATIONS

3.5.1 General

Laboratory tests are conducted on soil as well as rocks. The tests are generally varied based on the type and size of the structures and the geologic formations (soil and rock) in which the infrastructures are going to be built. In addition, the type of lab test parameters required will depend on the type of foundation and construction materials available in the project site.The purposes of laboratory testing on samples of soil and rock may be summarized as follows:

a) To describe and classify the samples which represent the actual materials on the site
b) To investigate the fundamental behavior of soil and rocks which determine engineering properties and appropriate method to be used in the analysis.
c) To obtain soil and rock parameters relevant to the technical objectives of the investigation

3.5.2 Selection of testing program

Laboratory testing of soils is a fundamental element of geotechnical engineering. The complexity of testing required for a particular project may range from a simple moisture content determination to specialized strength and stiffness testing. Since testing can be expensive and time consuming, the geotechnical engineer should recognize the project's issues a head of time so as to optimize the testing program, particularly strength and consolidation testing. Detail of selection program is presented in sub topic 2.5.2 above.

3.5.3 Sampling, transportation and storage of samples for laboratory tests

An important feature of a geotechnical laboratory is the provision of good facilities for handling, storing and inspecting samples. All samples entering the laboratory should be registered and receipts issued. Details on this issue are presented in sub topic 2.5.3. above.

3.5.4 Visual examination and description of laboratory samples

Introduction
Description of samples of soil and rock tested in the laboratory forms an important part of the record of the test results. Such descriptions should be included on the laboratory work sheet(s). Descriptions of samples made in the laboratory should be compared with the equivalent field descriptions and any anomalies resolved. Detail visual examination and description in the laboratory is presented in sub topic 2.5.4 above.

3.5.5 Tests on soil

Laboratory tests are used to determine design parameters and to complement field observations, field testing and back analysis of the behavior of existing structures that have been monitored. Detail of laboratory tests on soil is presented in sub topic 2.5.5 above.

3.5.6 Laboratory testing for rocks

Laboratory rock testing is performed to determine the strength and elastic properties of intact specimens and the potential for degradation and disintegration of the rock material. The derived parameters are used in part for the design of rock fills, cut slopes, shallow and deep foundations, tunnels, and the assessment of shore protection materials (rip-rap). Deformation and strength properties of intact specimen said in evaluating the larger-scale rock mass that is significantly controlled by joints, fissures, and discontinuity features (spacing, roughness, orientation, infilling), water pressures, and ambient geostatic stress state.

Common laboratory tests for intact rocks include measurements of strength (point load index, compressive strength, Braziliantest, directshear), stiffness (ultrasonics, elastic modulus), and durability (slaking, abrasion).

3.5.6.1 Strength tests

The laboratory determination of intactrock strength is accomplished by the following tests: point load index, unconfined compression, triaxial compression, Braziliantest, and directshear. The uniaxial (or unconfined) compression test provides the general reference value, having a respective analogy with standard tests on concrete cylinders. The uniaxial compressive strength ($q_u = F_u$) is obtained by compressing a trimmed cylindrical specimen in the longitudinal direction and taking the maximum measured force divided by the cross-sectional area. The point load index serves as a surrogate for the UCS and is a simpler test in that irregular pieces of rock core can be used. A direct tensile test requires special end preparation that is difficult for most commercial labs; therefore tensile strength is more often evaluated by compression loading of cylindrical specimens across their diameter (known as the Brazilian test). Direct shear tests are used to investigate frictional characteristics along rock discontinuity features

3.5.6.2 Durability

The evaluation of rock durability becomes an issue when the materials are to be subjected to the natural elements, seasonal weather, and repeated cyclesof temperature (e.g., flowing water, wetting and drying, wave action, freeze and thaw, etc.) in its proposeduse.Tests to measure durability depend on the type of rock, on its use in construction, and on the elements to which the rock will be subjected. The bases for durability tests are empirical and the results produced are an indication of the rock's resistance to natural processes; the rock's behavior in actual use may vary greatly from the test results.These tests, however, provide reasonably reliable tools for quality control. The suitability of various types of rock for different uses should, in addition to these test results, depend on their performance in previous applications. An example of the use of rock durability tests is in the evaluation of shale in rockfill embankments.

3.5.6.3 Deformation characteristics of intact rocks

The stiffness of rocks is represented by an equivalent elastic modulus at small-to intermediate-strains and **Ultrasonic Testing**

Frequently needed and recommended laboratory test types along with the type of the structures with their parameters needed to be evaluated are presented below on the table 3-13. Detail proposes, summarized testing procedures, standards and remarks on each test types are presented on appendix-I.

Table 3-13: Summary of recommended laboratory and field test along with the structures of small scale Irrigation projects

Structure types	Engineering Evaluations	Required Information for Analyses	LaboratoryTesting
Micro dam and associated structures such as spill way, intake, micro dam axis, reservoir area : Embankments and Embankment Foundations	-settlement (magnitude & rate) -bearing capacity -slope stability -lateral pressure -internal stability -borrow source evaluation (available quantity and -quality of borrow soil) -required reinforcement	-subsurface profile (soil, -ground water, rock) -compressibility -parameters -shear strength -parameters -unit weights -time-rate consolidation parameters -horizontal earth pressure -coefficients -interface friction -parameters -geologic mapping including orientation and characteristics of rock discontinuities shrink/swell/degradation of soil and rock fill	-1-D Oedometer -triaxial tests -direct shear tests -grain size distribution -Atterberg Limits -organic content -moisture-density relationship -hydraulic conductivity -geosynthetic/soil testing -shrink/swell -slake durability -unit weight -UCS for rocks -Modulus of elasticity
Canal, trenches, pond areas Excavations and Cut Slopes	-slope stability -bottom heave -liquefaction -dewatering -lateral pressure -soil softening/progressive failure -pore pressures	-subsurface profile (soil, ground water, rock) -shrink/swell properties -unit weights -hydraulic conductivity -time-rate consolidation parameters -shear strength of soil and rock (including discontinuities) -geologic mapping including	-hydraulic conductivity - grain size distribution - Atterberg Limits -triaxial tests - direct shear tests -moisture content -rock uniaxial compression test & intact rock modulus -point load strength test

Structure types	Engineering Evaluations	Required Information for Analyses	LaboratoryTesting
		orientation and characteristics of rock discontinuities	
Borrow area	Clay: Suitability for construction Volume of the borrow material (embankment materials)	Thickness of suitable layer Engineering property of the soil Shear strength of soil Shrink /swell properties of the soil	-hydraulic conductivity, grain size distribution, Atterberg Limits, triaxial tests, direct shear tests, moisture content, Erodibility, consolidation, compaction, chemical analysis, free swell
	Sand (fine aggregate)	Volume, Well grading or sorted, Organic content	Specific gravity, Sieve analysis, Soundness, Mineralogical analysis
	Gravel (coarse aggregate)	Durability Void Strength	Specific gravity, Sieve analysis, Water absorvation, Abration test, soundness, Mineralogical analysis
	Rock	Durability, Strength, Void, weathering	The tests as that of aggregate

NB: as per the standards

3.6 ENGINEERING GEOLOGICAL MAPPING

3.6.1 Introduction

An engineering geological map is a type of geological map which provides a generalized representation of all those components of a geological environment of significance in land use planning, in design, construction and maintenance as applied to civil engineering.

The purpose of engineering geology map is to provide basic information for planning, design, construction and maintenance of Micro dam and associated structures. Such information is needed to assess the feasibility of the proposed engineering undertaking, and for the selection of the most appropriate type and method of construction, to ensure the stability of a structure in its natural setting, and to aid the performance of necessary maintenance.

An engineering geological map should fulfill the following requirements:

- It should describe the objective information necessary to evaluate the engineering geological features involved in planning, in the selection of both a site and the most suitable method of construction.
- It should make it possible to foresee the changes in the geological situation likely to be brought about by a proposed undertaking and to suggest any necessary preventive measures.
- It should present information in such a way that it is easily understood by professional users who may not be geologists.

Geological features represented on engineering geological maps of Micro dam and associated structures sites are:

- The characters of the rocks and soils,
- Hydrogeological conditions
- Geomorphological conditions
- Geodynamic phenomena

Engineering geological maps should include interpretative cross-sections and an explanatory text and legend.

Engineering geological maps for the propose of Micro dam site characterization:

- Need to contain information in regard the influence of geological environment to the planned Micro dam and vis versa.
- The maximum area to be covered on the map not more than 500m upstream and downstream.
- The scale of map for micro Dam site is not less than 1: 5,000 (as per the prepared scale of the the project's structural design)

Geological mapping, description and classification of rocks and soils for engineering geological mapping of Micro dams and associated structures should follow similar proceedures as mentioned for diversion projects in this guide line above.

3.7 GEOLOGY AND GEOTECHNICAL ENGINEERING REPORTS OF MICRO DAMS

3.7.1 General

Upon completion of the field investigation and laboratory testing program, the geotechnical engineer will compile, evaluate, and interpret the data and perform engineering analyses for the design of foundations, embankments, and other required facilities. Additionally, the geotechnical engineer will be responsible for producing a report that presents the subsurface information obtained from the site investigations and provides specific technical recommendations. This chapter provides guidelines and recommendations for developing a geotechnical report.

Generally, one or two types of reports will be prepared: A geotechnical investigation (or data) report and a geotechnical design report. The choice depends on the requirements of the client and the agreement between the geotechnical engineer and the designer. The need for multiple types of reports on a single project depends on the project size, phasing and complexity and preparation of two reports is very commony especially when the site investigation drilling work is subcontracted some other companey.

3.7.2 Geotechnical investigation reports

Geotechnical investigation reports present site-specific data and have three major components:
Background Information: The initial sections of the report summarize the geotechnical engineer's understanding of the structures for which the report is being prepared and the purposes of the geotechnical investigation. This section would include information on loads, deformations and additional performance requirements. This section also presents a general description of site conditions, geology and geologic features, drainage, ground cover and accessibility, and any peculiarities of the site that may affect the design.

The second part of the investigation report documents the scope of the investigation program and the specific procedures used to perform this work. These sections will identify the types of investigation methods used; the number, location and depths of borings, excavation of test pits and insitu tests; the types and frequency of samples obtained; the dates when the field investigation was performed; the subcontractors used to perform the work; the types and number of laboratory tests performed; the testing standards used; and any variations from conventional procedures.

Data Presentation: This portion of the report, generally contained in appendices, presents the data obtained from the field investigation and laboratory testing program, and typically includes final logs of all borings, excavated testpits, and piezometer or well installations, water level readings, data plots from each in-situ test hole, summary tables and individual data sheets for all laboratory tests performed, rock core photographs, geologic mapping data sheets and summary plots, subsurface profiles developed from the field and laboratory test data, as well as statistical summaries. Often, the investigation report will also include copies of existing information such as boring logs or laboratory test data from previous investigation data of the project site.

The intent of a geotechnical investigation report should be to document the investigation performed and present the data obtained. The report should include a summary of the subsurface and lab data.

Interpretation and recommendations on the index and design properties of soil and rock should also be included. This type of report typically does not include interpretations of the subsurface conditions and design recommendations.The geotechnical investigation report is some times used when the field investigations are subcontracted to a geotechnical consultant, but the data interpretation and design tasks are to be performed by the owner's or the prime consultant's in-house geotechnical staff.

1.0 INTRODUCTION
2.0 SCOPE AND OBJECTIVE OF WORK
3.0 SITE DESCRIPTION
4.0 FIELD INVESTIGATION PROGRAM & IN-SITUTESTING
5.0 DISCUSSION OFLABORATORY TESTS PERFORMED
6.0 SITE CONDITIONS, GEOLOGICal SETTING, & TOPOGRAPHIC INFORMATION
7.0 SUMMARY OF SUBSURFACE CONDITIONS AND SOIL PROFILES
8.0 DISCUSSION OF FINDINGS, CONCLUSIONS, AND RECOMMENDATIONS
9.0 REFERENCES

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Appendix II- Test Boring Logs and Core Logs With Core Photographs
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Appendix IV-Geophysical Survey Data
Appendix V –Field Permeability Test
Appendix VI –LaboratoryTest Results
Appendix VII- Existing Information

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3.7.3 Geotechnical design reports

A geotechnical design report typically provides an assessment of existing subsurface conditions at a project site, presents, describes and summarizes the procedures and findings of any geotechnical analyses performed, and provides appropriate recommendations for design and construction of foundations of the structures and other geotechnical issues of require dfacilities. Unless a separate investigation report has previously been developed, the geotechnical design report will also include documentation of any subsurface investigations performed and a presentation of the investigation data as described insection geotechnical Investigation report above.

An example *Table of Contents* for a geotechnical design report of Micro Dam is presented below.

Since the scope, site conditions, and design/construction requirements of each project are unique, the specific contents of a geotechnical design report must be tailored for Micro dam and associated structure project. In order to develop this report, the geotechnical engineer who is going to prepare the report must possess detailed knowledge of thes structures and the sub surface conditions.In general; however, the geotechnical design report must address all the geotechnical issues that may be anticipated on a project. The report must identify each soil and rock unit of engineering significance, and must provide recommended design parameters for each of these units. This requires a summarization and analysis of all factual data to justify the recommended index and design properties. Groundwater conditions are particularly important for both design and construction and, accordingly, they need to becarefully assessed and described. For every project, the subsurface conditions encountered in the site investigation need to be compared with the geologic setting to better understand the nature of the deposits and to predict the degree of variability between borings, auguring or pitting.

Each geotechnical design issue must be addressed in accordance with the methodology described in subsequent topics of this guide manual, and the results of these studies need to be concisely and clearly discussed in the report. Of particular importance is an assessment of the impact of existing subsurface conditions on construction operations, phasing and timing. Properly addressing these items in the report can preclude change-of-conditions claims. Examples include but are not limited to be presented below as follows:

Recommendations should be provided for solution of anticipated problems.

1.0 INTRODUCTION

- Back ground/ Overview previous works of the project
- Objective and Scope of the project
- Location and accessibility of the project (each structure)
- Climate vegetation and current land use

2.0 REGIONAL AND SITE GEOLOGY

- Regional geology of the area
- Site geology
- Geomorphology
- Geologic structures of the area
- Hydrogeology

- Engineering geological map of the area showing locations of Boreholes, Test pits trenches, geophysical survey lines and profiles through the infrastructure locations.

3.0 METHODOLOGY OF THE INVESTIGATION

- Planning
- Type of investigation methods and testing's
- Laboratory test types
- etc

4.0 FINDINGS

Micro dam, Canal etc and their sections as well as associated structural unit

- Review
- Site investigations
- Laboratory tests
- Analysis of both site investigation as well as laboratory tests in line with the infrastructure under considerations
- Construction material assessments (suitability, quantity, haulage distance, etc)
    a. Impervious materials (clay core, fill, canal lining, etc)
    b. Embankment (for micro dam body)
    c. Rock sources (to use as rock fill, for production of aggregate, masonry work, rip-rap)
    d. Sand as Filter/Transition materials and concrete making
    e. water
  - etc

5.0 FOUNDATION RECOMMENDATIONS

- Foundations bearing condition of micro dam site and associated pond or other structure
- Foundation depth and methods of excavation of the micro dam site and associated pond, canal or other structure
- Foundation permeability conditions of dam site, pond or other structure
- etc

6.0 CONCLUSIONS AND RECOMMENDATIONS

- Conclusions
- Recommendations

<u>Appendixes:</u>

Appendix I- Borehole/ test pits location Plan and subsurface Profiles
Appendix II- Test Boring Logs and CoreLogs With Core Photographs
Appendix II- Field Insitu tests
Appendix IV-Geophysical Survey Data
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4 INSTABILITY OR LAND SLIDE PROBLEMS

4.1 RECOGNITION AND IDENTIFICATION OF MASS MOVEMENT

In dealing with mass movement assessment an appropriate way to preliminary investigation is an adequate analysis of the regional geology. Typical vulnerable location include, steep slopes, cliffs or banks undercut by stream areas of drainage concentration and seepage zones, hummocky ground and areas of fracture and fault concentration. The most common causes of a large number of slides on steep slopes is residual or colluvial soil sliding on bedrock surface. Loose unconsolidated soils cannot maintain as steep a slope as the underlying bedrock and therefore in a delicate balance; any of several factors such as heavy rainfall, vibration by earthquakes, blasting, heavy traffic, or an excavation at the toe can trigger a movement.

- Interpretation of aerial photos has proved to be an extremely powerful tool for recognition and delineation of mass movements. No other technique can provide a three dimensional model of the terrain from which, the interrelations of topography, drainage, surface cover, geologic materials and human activities on the landscape become so clear.
- AP'S (Aerial photographs) present a good over all prospective (synoptic view) of large area.
- Boundaries of existing slides can easily be detected;
- Tracing of surface and near surface drainage canals is easy;“Important relations in drainage, topography, geology and rnan-made elements, thatseldomly are correlated properly on the ground, become obvious on AP'S;
- A moderate vegetation cover is less disturbing on AP'S than it is on the ground;
- Soil and rock formation can easily be distinguished;GI Continuity or repetition of features is emphasized;
- Field investigation program can be planned effectively;
- Comparison of old and recent AP'S can reveal the development of mass movementareas
- AP' S can be studied in any place at any time.

Diagnostic patterns of mass movements on AP'S

- Land masses under cut by streams;
- Steep slopes having large masses of loose soil and rock;
- Sharp line of break at the scarp (head end) or presence of tension cracks (or both);
- Hummocky surface;
- Unnatural topography such as spoon shaped trough in the terrain;
- Seepage zones
- Elongated undrained depressions on the slope;
- Closely spaced drainage channels;
- Accumulations of debris in drainage canals or valleys; appearance of light tones where
- vegetation and drainage have been re-established;
- Distinctive change in topographic tones from lighter to darker, the darker tonesindicating higher moisture constant; Inclined fences and trees (only in very specialcases).

4.1.1 Recognition of old and recent mass movements

Old slides: the presence of hummocky ground and may even the presence of a scarp surfacemay not be very distinct are often indication of an existing or old mass movement;the older the movement is the more established drainage and vegetation become; sodrainage and vegetation may help to determine the relative age and stability of the oldmass movement; once an old mass movement is detected,it can be seen as proof that thearea has been instable in the past; it can serve as warning that new disturbances (eithernatural or man made) may induce new movements, however, it can not be assumed without further investigation that every old sliding areais by definition instable at present; in detecting old mass movements (such as slides and flows) the use of AP's is aparticularly powerful tool.

Recent mass movements: usually they are rather easily detectable on AP's due to theirshape and the light tone of bare soil /rock/,when the vegetation cover is ripped off; a massmovement in general may indicate that a more stable condition (flattening of the slope) hasbeen achieved, but on the other hand, especiallyin unconsolidated deposits, the materialspresent in the scarp face remain in an unstablecondition- the scarp face rapidlyretrogrades uphill by continued slumpinguntil a more stable condition results; a recentslide should be investigated very carefully to judge the probability of the occurrence offuture movement; the most significant *sign* of further movements the presence of cracksin the crown of the slide; such cracks, especially if an appropriate scale is used, can bedetected on air photos.

The interpretation of mass movement from aerial photos,may preferably be done by anexperienced Engineering geologist or Geomorphologist.

4.2 CLASSIFICATION OF MASS MOVEMENT

The best classification system is the one by D.J \/arnes, which is based on the type of movement and type of material. (See Table 4-1)

Fall: Mass in motion travels most of distancethrough air. It includes free-fallmovement by leaps and bounces, and rolling of bedrock.

Topples: Movements due to force that causesan overturning movement about apivot point below the center of gravity of the unit. If unchecked it will result in fall orslide.

Slide: Movement involves shear displacement along one or several surfaces, or withina relatively narrow zone, which are visible or may reasonably inferred. This includethe following:

- **Rotational**, movement due to forces that cause a turning moment about a point above the center of gravity of the unit surface of rapture concave upward.
- **Translational,** movement predominantly along more or less planar or gently undulatory surfaces. Movement is frequently structurally controlled bysurfaces of weakness, such as faults, joints, bedding planes etc.

Flows: Flows could be in bedrock or in soil. Flows in bed rock Include spatiallycontinuous deformation and surficial as well as deep creep, involving extremely slowand generally non-accelerating differential movements along relatively intact units.Flows in soil include movement with displaced mass such as the form taken bymoving material to the apparent distribution of velocities and displacements resemblethose of viscous fluids.

Table 4-1: Classification of mass movement (D.J.Vames)

<table>
<tr><th colspan="2" rowspan="3">Types of movement</th><th colspan="3">Types of materials</th></tr>
<tr><th rowspan="2">Bed rock</th><th colspan="2">Engineering soils</th></tr>
<tr><th>Mostly coarse</th><th>Mostly fine</th></tr>
<tr><td colspan="2">Falls</td><td>Rock fall</td><td>Debris fall</td><td>Earth fall</td></tr>
<tr><td colspan="2">Topples</td><td>Rock topple</td><td>Debris topple</td><td>Earth topple</td></tr>
<tr><td rowspan="2">Slides</td><td>Rotational</td><td>Rock slamp</td><td>Debris slamp</td><td>Earth slamp</td></tr>
<tr><td>Transitional</td><td>Rock slide</td><td>Debris slide</td><td>Earth slide</td></tr>
<tr><td colspan="2">Flows</td><td>Rock flow (deep creep)</td><td>Debris flow</td><td>Earth flow (soil creep)</td></tr>
<tr><td colspan="2">Complex</td><td colspan="3">Combination of two or more main types of movement</td></tr>
</table>

In addition to the reservoir body, more attention should be given to the abutments in termsof stability. The formation of a reservoir upsets the groundwater regime and represents anobstruction to water flowing downhill. The greatest change involves the raising of thewater table. Some rocks, which formerly were not within the zone of saturation, may then become unstable and fall as saturated material is weaker than unsaturated. This can lead to slumping and sliding on the flanks of the reservoir. Landslides, which occur after areservoir is filled reduces its capacity.

4.3 CAUSES OF MASS MOVEMENTS

The followings are some of the causes of mass movements.

A. Removal of lateral support is one of the most common factors leading to instability.
This is caused by,

- erosion by streams and rivers
- Construction of cuts, quarries, pits and canals.

B. Surcharge which results from both natural and human agencies is caused by,

- Weight of rain, hail. .. water from springs
- Seepage pressure of percolating water
- Construction of fill
- Waste pits
- Weight of water from leaking pipelines, reservoirs sewers and canals.

C. Transitory earth stress:

- Earthquakes
- Weight of traffic (trains and trucks)
- Vibrations from blasting, machinery, thunder adjacent slope failure

D. Removal of underlying support:

- Under cutting of banks by rivers
- Subterranean erosion is which soluble material such as carbonates, salt, gypsum isremoved and granular material beneath firmer soil is worked out.
- Mining or similar activities

E. Composition materials that are inherently weak or may become weak on changes inwater content, e.g. especially organic materials, sedimentary clays and shale'sdecomposed rocks, volcanic tuffs and materials that are composed dominantly of softminerals such as mica, talc, serpentine.

Texture: Very important in this aspect the roundness of the individual grains whichstrongly influences cohesive strength, compressibility and internal friction.

F. Gross structure and slope geometry

- Discontinuities such as faults bedding planes foliation, cleavage, joints, slickensides, brecciated zones.
- Massive beds over weak or plastic material
- Strata inclined towards free face
- Alteration of permeable beds (sandstone, conglomerates etc.) and weakimpermeable beds (clays, shales)
- Slope orientation (steepness, attitude)

G. Deep scared deformation

4.4 REMEDIAL MEASURES

The possible methods of stabilization fall within three groups:

A. Changing the geometry of the land slipped mass
B. Drainage
C. Giving support by reinforcement

A. Changing the geometry of the land slipped mass

The stability of a slope depends upon the inclination of the slope in relation to the strength of materials, the strength and orientation of discontinuities and the groundwater conditionswithin the slope. It is clear that a reduction in the angle of the slope or the removal ofmaterial from the top of the slope, will improve the factor of safety of the slope. Although removal of material seems an easy solution to landslips it may posse considerable practical problems. Often after small slides the whole of the landslide is removed and replaced bydense granular permeable rock fill.

B. Drainage

Slope may be drained by the introduction of shallowly inclined drain hole. The drain holesare usually drilled into slopes by rotary drilling methods. The hole is lined with a slottedtube and often this tube is itself surrounded by a solid ceramic filter material if theborehole is in soil. Occasionally drainage may be undertaken by pumped wells.

Pumped wells are seldom used as a permanent means of slope stabilization because of theoperation costs. However, they may be used as a temporary remedy, modifying thepumping rate to find the correct reduction of the water table level to stop slope movementand hence driving a drainage tunnel to achieve this required reduction on a permanentbasis. This is particularly useful technique in large open mines where drills are readilyavailable.

C. Support

Support can be given by reinforcement or by the application of force. Attempts haveoccasionally been made to add reinforcement by loading the toe of the landslide withdense granular material free draining and by constructing a retaining wall at the root of theland slide material.

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APENDECIES

APPENDIX I: Laboratory test types along with their propose and proceedures of standard testings

Classification (Index) tests

Index properties are used to characterize soils and determine their basic properties such as moisture content, specific gravity, particle size distribution, and consistency and moisture-density relationships. Exact and complete descriptions and procedures of each test, presented in the standards ASTM, BS, DIN, etc.However, in order to give an overview, the tests are discussed brifelybelow as follows:

Prupose and summerized test proceedures of moisture content

Laboratory test	*prupose*	*summarized of Test procedures and Standards*	*Remarks*
Moisture content	To determine the amount of water present in a quantity of soil in terms of its dry weight and to provide general correlations with strength, settlement, workability and other properties.	Oven-dry the soil at a temperature of 110±5oC to a constant weight (evaporate free water); this is usually achieved in 12 to 18 hours. Standards: AASHTO T265 ASTM D 4959 BS1377:1975 test 1	The moisture content of soils, when combined with data obtained from other tests, produces significant information about the characteristics of the soil. For example, when the in situ moisture content of a sample retrieved from below the phreatic surface approaches its liquid limit, it is an indication that the soil in its natural state is susceptible to larger consolidation settlement.

Pruposed and summerized test proceedures of Seive analysis

Lab. Test	*prupose*	*summarized of Test procedures and Standards*	*Remarks*
Sieve Analysis	To determine the percentage of various grain sizes. The grain size distribution is used to determine the textural classification of soils (i.e., gravel, sand, siltyclay, etc.) which inturn is useful in evaluating the engineering characteristics suchas permeability, strength, swelling potential, and susceptibility to frostaction.	Wash a prepared representative sample through a series of sieves (screensThe amount retained on each sieve is collected dried and weighed to determine the percentage of material passing that sieve size. **Standards:** AASHTO T88 ASTM D 422, 1140 BS1377:1975 test 7	Obtaining a representative specimen is an important aspect of this test. When samples are dried for testing or “washing,” it may be necessary to break up the soil clods. Care should be made to avoid crushing of soft carbonate or sand particles. If the soil contains a substantial amount of fibrous organic materials, these may tend to plug the sieve openings during washing. The material settling over the sieve during washing should be constantly stirred to avoid plugging.

Prupose and summerized test proceedures of Hydrometere analysis

Lab. Test	Prupose	summarized Test procedures and Standards	Remarks
Hydrometer Analysis	To determine distribution (percentage) of particle sizes smallerthan No.200sieve (<0.075 mm) and identify the silt, clay, and colloids percentages in the soil.	Soil passing the No.200 sieve is mixed with dispersant and distilled water and placed in a special graduated cylinder in a state of liquid suspension. The specific gravity of the mixture is periodically measured using a calibrated hydrometer to determine the rate of settlement of soil particles.The relative size and percentage of fine particles are determined based on Stoke's law for settlement of idealized spherical particles. **Standards:** AASHTO T88 ASTMD 1140 BS1377:1975 test 7	The principal value of the hydrometer analysis is in obtaining the clay fraction (percent finer than 0.002 mm). This is because the soil behavior for a cohesive soil depends principally on the type and percent of clay minerals, the geologic history of the deposit, and its water content rather than on the distribution of particle sizes.

Prupose and summerized test proceedures of specific gravity

Lab. Test	prupose	summarized Test procedures and Standards	Remarks
Specific Gravity	To determine the specificgravity of the soilg rains.	The specific gravity is determined as the ratio of the weight of a given volume of soil solids at a given temperature to the weight of an equal volume of distilled water at that temperature, both weights being taken in air. **Standards:** AASHTO T100 ASTMD 854 BS1377:1975 test 7	Some qualifying words like *true, absolute, apparent, bulk or mass,* etc. are sometimes added to "specificgravity". These qualifying words modify the sense of specific gravity as to whether it refers to soil grains or to soil mass.The soil grains have permeable and impermeable voids inside them. If all the internal voids of soil grains are excluded for determining the true volume of grains, the specific gravity obtained is called *absolute* or *true* specificgravity. A value of specific gravity is necessary to compute the void ratio of a soil, it is used in the hydrometer analysis, and it is useful to predict the unit weight of a soil. Occasionally, the specific gravity may be useful in soil mineral classifications; e.g., iron minerals have a large rvalue of specific gravity than silica.

Prupose and summerized test proceedures ofAtterberg limits

Lab. Test	prupose	summarized Test procedures and Standards	Remarks
Atterberg Limits	To describe the consistency and plasticity of fine-grainedsoils with varyingdegrees of moisture.	For the portion of the soil passing the No.40 sieve, the moisture content is varied to identify three stages of soil behavior interms of consistency.These stages are known as the liquid limit (LL), plastic limit (PL) and shrinkage limit (SL) of soils. The ***liquidlimit*** **(LL)** is defined as the water content at which 25 blows of the liquid limit machine closes a standard groove cut in the soil pat for a distance of 12.7cm. An alternate procedure in Europe and Canada uses a fall cone device to obtain better repeatability. The ***plasticlimit*** **(PL)** is as the watercontent at which athread of soil, when rolled down to a diameter of 3mm, will crumble. The ***shrinkagelimit*** **(SL)** is defined as that water content below which no further soil volume change occurs with further drying. **Standards:** AASHTO T89, T90 ASTMD 4318 BS1377:1975 test 2,3,4,5	The Atterberg limits provide general indices of moisture content relative to the consistency and behavior of soils. The LL defines a liquid/semi-solid change, while the PL is a solids boundary. The difference is termed the *plasticity index* (PI=LL-PL). The *liquidity index* is LI= (w-PL)/PI is an indicator of stress history; LI ₋ 1 for normally consolidated (NC) soils and LI ₋ 0 for over-consolidated (OC) soils. By and large, these are approximate and empirical values. **PI ranges** <1 Non plastic 1 – 7 slightly plastic 7- 17 moderately plastic 17- 35 highly plastic >35 extremely plastic **Compressibility range** <35 low 35 – 50 medium >50 high

Prupose and summerized test proceedures of Compaction

Lab. Test	Prupose	summarized Test procedures and Standards	Remarks
Moisture density relationship (compaction)	To determine the maximum dry density attainable under a specified nominal compaction energy for a given soil and the (optimum) moisture content corresponding to this density.	Compaction tests are performed using disturbed, prepared soils with or without additives. Normally, soil passing the No.4 sieve is mixed with water to form samples at various moisture contents ranging from the dry state to wet state. These samples are compacted in layers in a mold by a hammer in accordance with specified nominal compaction energy. Dry density is determined based on the moisture content and the unit weight of compacted soil. A curve of dry density versus moisture content is plotted in and the maximum ordinate on this curve is referred to as the maximum dry density (Dmax). The water content at which this dry density occurs is termed as the optimum moisture content (OMC). **Standards:** AASHTO T99, T180 ASTMD 698, D1557 BS1377:1975 test 2,3,4,5	In the construction of highway embankments, earth dams, retaining walls, structure foundations and many other facilities, loose soils must be compacted to increase their densities. Compaction increases the strength and stiffness characteristics of soils. Compaction also decreases the amount of undesirable settlement of structures and increases the stabilityof slopes and embankments. The density of soils is measured as the unit dryweight, (D, (weight of dry soil divided by the bulk volume of the soil). It is a measure of the amount of solid materials present in a unit volume.

Prupose and summerized test proceedures of organic contents analysis

Lab. Test	prupose	summarized Test procedures and Standards	Remarks
Organic Content of Soils	To help classify the soil and identify its engineering characteristics .	Oven-dried (at110±5^{o}C) samples after determination of moisture content are further gradually heated to 440^{o}C which is maintained until the specimen is completely ashed (no change in mass occurs after a further period of heating).The organic content is then calculated from the weight of the ash generated. **Standards:** AASHTO T194 ASTMD 2974 BS1377:1975	Organic soils are those formed throughout the ages at low-lying sediment –starved areas by the accumulation of dead vegetation and sediment. Top soils are very recently formed mixtures of soil and vegetation that form part of the food chain. Top soils are not suitable for use in construction and there fore its organic content is not usually determined Organic materials affect the behavior of soils in varying degrees.The behavior of soils with low organic contents (<20% by weight) generally are controlled by the mineral components of the soil. When the organic content of soils approaches 20%, the behavior changes to that of organic, or peaty soils. The consolidation characteristics, permeability, strength and stabilization of these soils are largely governed by the properties of organic materials. Thus it is important to determine the organic content of soils. It is not sufficient to simply label a soil as "organic"without showing the organic content.

Laboratory tests for determination the strength properties of soil

Design and analysis of shallow and deep foundations, excavations, earth retention structures, and fills and slopes require a thorough understanding of soil strength parameters.Thes election of strength parameters needed and the corresponding types of tests to be performed vary depending on the type of construction, the foundation design, the intensity, type and duration of loads to be imposed, and soil materials existing at the site.

The shear strength should be determined by a combination of both field and laboratorytests. Lab tests provide reference strengths under controlled boundaries and loading. However, limited quality samples are obtained from the field, particularly for sandy materials. The interpretation of strength from in-situ tests in sands and clays is important.

Commonly used laboratory tests for clays include the unconfined compression (UC) and unconsolidated undrained tests(UU), however, these donot attempt to replicate the ambient stress regime in the ground prior to loading and therefore can only be considered as index strengths. Preferably, the consolidated triaxial shear and direct shearbox tests can be used in conjunction with consolidation/oedometer tests in anormalized stress history approach (Ladd &Foott, 1974; Jamiolkowski, et al. 1985).

Both undisturbed and remolded or compacted samples are used for strength tests. Where soils are to be disturbed and remolded, compacted or stabilized specimens are tested for strength determination at specified moisture contents and densities. These may be chosen on the basis of design requirements or the in-situ density and misture content of soils. Where obtaining undisturbed samples is not practical (i.e., sandy and gravelly soils), specimens remolded close to their natural moisture content and density are prepared for testing.

Prupose and summerized test proceedures of Un confined compresive strength (UCS)

Lab. Test	Prupose	summarized Test procedures and Standards	Remarks
Unconfined Compressive Strength of Soils	To determine the undrained shear strength (c_u) of clay and siltyclay soils.	The soil specimens are tested without any confinement or lateral support (F_3=0). Axial load is rapidly applied to the sample to cause failure. At failure the total minor principal stress is zero (F_3=0) and the total major principal stress is F_1. The maximum measured force over the sample area is q_u and referred to as the unconfined compression strength. Since the undrained strength is independent of the confining pressure, $c_u=q_u/2$ **Standards:** AASHTO T208 ASTMD 2166 BS1377:1975 test 2,3,4,5	The determination of unconfined compressive strength of undisturbed, remolded or compacted soils is limited to cohesive or naturally or artificially cemented soils. Application of this test to non-cohesive soils may result in under estimation of the shear strength. The test is in expensive and requires a relatively short period of time to complete.However,due to the absence of lateral pressure sand lack of control over porepressures, it has major in accuracies. The stress-strain curves and failure modes observed during testing provide an index value of the soil properties in addition to strength. For example,an ill-defined failure or yielding of the sample signifies a relatively soft, fat clay, while a sudden brittle failure indicates that of a desiccated clay or cemented material.

Propose and summerized test proceedures of triaxial strength

Lab. Test	Prupose	summarized of Test procedures and Standards	Remarks
Triaxial Strength	To determine strength characteristics of soils including detailed information on the effects of lateral confinement, porewater pressure, drainage and consolidation. Triaxial tests provide a reliable means to determine the friction angle of natural clays & silts, as well as reconstituted sands. The stiffness (modulus) at intermediateto large strains can also be evaluated.	Test samples are typically 35 to 75 mm in diameter and have a height to length ratio between 2 and 2.5. The sample is encased by a thin rubber membrane and placed inside a plastic cylindrical chamber that is usually filled with water or glycerine. The sample is subjected to a total confining pressure (F_3) by compression of the fluid in the chamber acting on the membrane. A back pressure (u_o) is applied directly to the specimen through a port in the bottom pedestal. Thus, the sample is initially consolidated with an effective confining stress: $F_3r=(F_3-u_o)$. (Note that air should not be used as a compression medium). To cause shear failure in the sample, axial stress is applied through a vertical loading ram (commonly called *deviator stress*=F_1-F_3). Axial stress may be applied at a constant rate (strain controlled) or by means of a hydraulic press or dead weight increments or hydraulic pressure (stress controlled) until the sample fails. The axial load applied by the loading ram corresponding to a given axial deformation is measured by approving ring or electronic load cell attached to the ram. Connections to measure drainage in to or out of the sample, or for porewater pressure are also provided. Deflections are monitored by dialindicators, LVDTs, or DCDTs. **Standards:** AASHTO T 296, T297 ASTM D 2850, D4767 BS1377:1975	There are five types of triaxial tests Undrained Unconsolidated (UU test) Consolidated Undrained (CU test) Consolidated Drained (CD test) Consolidated Undrained with pore pressure measurement (C6U6) Cyclic Triaxial Loading Tests (CTX) In a UU test, the samples are not allowed to drain or consolidate prior to or during the testing. The results of undrained tests depend on the degree of saturation (S) of the specimens. Where S=100%, the test results will provide a value of undrained shear strength (s_u), however, the test is affected by sample disturbance and rate effect (Ladd, 1991). This test is not applicable for granular (S=100%) soils. The (CU) test with porewaterpressure measurements is the most useful as it provides a direct measure of the undrained shear strength (s_u), for triaxial compressive mode, a swell as the important effective stress parameters (crandNr). The CD tests also provide the parameters Cr and Nr. Cyclic triaxial tests are used for projects with repeated and/or cyclic loading, resilient modulus determinations, and/or liquefaction analysis of soils. In each of these tests, the specimen is initially consolidated to the effective vertical overburden stress ($F_{vo}r$) prior to shear.

Prupose and summerized test proceedures of direct shear

Lab. Test	*Prupose*	*summarized Test procedures and Standards*	*Remarks*
Direct Shear	To determine the shear strength of soils along apre-defined (horizontal) planar surface	The direct shear (DS) test is performed by placing a specimen into a cylindrical or square- shaped shear box which is split in the horizontal plane. A vertical (normal) load is applied over the specimen that is allowed to consolidate. While either the upper or lower part of the box is held stationary,a horizontal load is exerted on the other part of the box in an effort to shear the specimen on apredefined horizontal plane. The test is repeated at least three times using different normal stresses (FNr) The results are plotted in the form shear stress (J) vs horizontal displacement(*), and corresponding J vs FNr. The effective cohesion intercept and angle of internal friction values can be determined from this latter plot. **Standards:** AASHTO T236 ASTMD 3080 BS1377:1975 test 2,3,4,5	The DS test is the oldest and simplest form of shear test arrangement. It has several inherent shortcomings due to the forced plane of shearing: The failure plane is predefined and horizontal; this plane may not be the weakest. As compared to the triaxial test, there is little control over the drainage of the soil. The stress conditions across the soil sample are very complex. The distribution of normal stresses and shearing stresses over the sliding surface is not uniform; typically the edges experience more stress than the center. Due to this, there is progressive failure of the specimen, i.e., the entire strength of the soil is not mobilized simultaneously. In spite of the above shortcomings, the direct shear test is commonly used as it is simple and easy to perform. The device uses much less soil than a standard triaxial device, therefore consolidation times are shorter. The DS provides reasonably reliable values for the effective strength parameters, cr and Nr, provided that slow rates of testing are utilized. Repeated cycles of shearing along the same direction provide an evaluation of the residual strength parameters (crr and Nrr). The direct shear test is particularly applicable to those foundation design problems where it is necessary to determine the angle of friction between the soil and the material of which the foundation is constructed, e.g., the friction between the base of a concrete footing and underneath soil. In such cases, the lower box is filled with soil and the upper box contains the foundation material.

Prupose and summerized test proceedures of Clifornian bearing ratio (CBR)

Lab. est	Prupose	summarized Test procedures and Standards	Remarks
California Bearing Ratio (CBR)	To determine the bearing capacity of a compacted soil under controlled moisture and density conditions.	The test results are expressed in terms of a bearing ratio which is commonly known as the California Bearing Ratio (CBR). The CBR is obtained as the ratio of the unit load required to cause a certain depth of penetration of a piston into a compacted specimen of soil at some water content and density, to the standard unit load required to obtain the same depth of penetration on a standard sample of crushed stone (usually limestone). Typically soaked conditions should be used to simulate anticipated long-term conditions in the field. The CBR test is run on three identically compacted samples. Each series of the CBR test is run for a given relative density and moisture content. The geotechnical engineer must specify the conditions (dry, at optimum moisture, after soaking, 95% relative density, etc.) under which each test should be performed. **Standards:** AASHTO T193 ASTMD 4429, D1883	CBR is a practical bearing capacity test, yet provides only discrete point test data for evaluation. Most CBR testing is laboratory-based, thus the results will be highly dependent on the representativeness of the samples tested. The test results are used for highway, airport, parking lot and other pavement designs using empirical local or agency-specific methods (i.e., FHWA, FAA, and AASHTO).

Laboratory Permeability test

The hydraulic conductivity or permeability is an important flow property of soils.

Prupose and summerized test proceedures of permiability test of soil in laboratory

Lab. Test	prupose	summarized Test procedures and Standards	Remarks
Permeability of Soils	To determine the potential of flow of fluids through soils.	The ease with which a fluid passes through a porous medium is expressed in terms of coefficient of permeability (k), also known as hydraulic conductivity. There are two basic standard types of test procedures to directly determine permeability in the laboratory: (1) constant-head; and (2) falling-head procedures. In both procedures, undisturbed, remolded, or compacted samples can be used. The permeability of course materials is determined by constant head tests. The permeability of clays is normally determined by the use of a falling head permeameter. The difference between the two tests is that in the former, the hydraulic gradient of the specimen is kept constant, while in the latter, the head is allowed to decrease as the water permeates the specimen. Evaluations of soil permeability are obtained from time readings required for a measured volume of water to pass through the soil. **Standards:** AASHTO T215 ASTMD 2434, D5084	Permeability is one of the major parameters used in selecting soils for various types of construction. In some cases it may be desirable to place a high-permeability material immediately under a pavement surface to facilitate the removal of water seeping into the base or sub-base courses. In other cases, such as retention pond dikes, it may be detrimental to use high-permeability materials. Permeability also significantly influences the choice of backfill materials. Laboratory permeability tests produce reliable results under ideal conditions. Permeability of fine-grained soils can also be computed from one-dimensional consolidation test results, although these results are not as accurate as direct k measurements (e.g., Tavenas, et al. 1983).

Consolidation and swelling test

The one-dimensional consolidation test (or oedometer test) provides one of the most useful and reliable laboratory measurements for soil behavior. The test determines the compressibility parameters (Cc, Cs, Cr), stiffness in terms of constrained modulus (Dr = 1/mv), preconsolidation stress (Fpr), rate of consolidation (cv), creep rate (C"), and approximate value of permeability (k).

Prupose and summerized test proceedures of moisture content

Lab. Test	Prupose	summarized Test procedures and Standards	Remarks
One-Dimensional Consolidation	Determination of preconsolidation stress, compression characteristics, creep, stiffness, and flow rate properties of soils under loading.	The test is performed using a small 50-mm to 75-mm diameter thin specimen (25 mm thick) taken from an undisturbed sample. Selection of representative samples for testing is critical. Prepared samples are placed in a rigid-walled loading device called a consolidometer or oedometer. All loads and recorded deformations are in the vertical direction. The specimen is subjected to incremental loads, which are doubled after each equilibrium phase is reached (after tp corresponding to the end of primary consolidation). Tradition would use a 24-hour increment per load, although this is conservative. Alternatively, specimens can be loaded continuously with monitoring by load cells and porewater pressure transducers. Generally, it is desirable to perform an unload-reload cycle during the test, with the unloading initiated at a loading increment along the virgin portion of the consolidation curve. The unload-reload cycle provides a more reliable estimate of the recompression characteristics of the soil. **Standards:** AASHTO T216 ASTMD 2435	When saturated soil masses are subjected to incremental loads, they undergo various degrees of dimensional change. Initially, the incremental load is resisted and carried by the liquid phase of the soil, which develops excess porewater pressures (u) in the soil voids. Depending on the permeability and the availability of drainage layer(s) in contact with the soil, the liquids in the voids begin draining and continue to do so until the)u is dissipated. As the hydrostatic pressure decreases, a proportional amount of the incremental load is transferred to the solid portion of the soil. When the excess hydrostatic pressure reaches zero, the entire new load is carried by the soil's solids. This process is called primary consolidation. In granular, high-permeability soils, this transfer take place very quickly (since water can drain fast). In clays and low-permeability soils, primary consolidation takes a longer time, which can affect the long-term performance of structures supported by these soils. Time rate is expressed by the coefficient of consolidation (cv). The one-dimensional consolidation test is most commonly used for the determination of consolidation properties of soils. This test method assumes that dimensional change due to consolidation will take place in the vertical direction. This assumption is generally acceptable for stiff or medium, confined cohesive soils, but it is not true for soft soils or for soils that are not confined (i.e., bridge approaches). The data and the analysis produced from this test have proved to be reasonably reliable.

Prupose and summerized test proceedures of moisture content

Lab. Test	Prupose	summarized Test procedures and Standards	Remarks
Swell Potential of Clays	To estimate the swell potential of (expansive) soils	The swell test is typically performed in a consolidation apparatus. The swell potential is determined by observing the swell of a laterally-confined specimen when it is surcharged and flooded. Alternatively, after the specimen is inundated, the height of the specimen is kept constant by adding loads. The vertical stress necessary to maintain zero volume change is the swelling pressure. **Standards:** AASHTO T256 ASTMD 4546	Swelling is a characteristic reaction of some clay to saturation. The potential for swell depends on the mineralogical composition. While montmorillonite (smectite) exhibits a high degree of swell potential, illite has none to moderate swell characteristics, and kaolinite exhibits almost none. The percentage of volumetric swell of a soil depends on the amount of clay, its relative density, the compaction moisture and density, permeability, location of the water table, presence of vegetation and trees, and overburden stress. Swelling of foundation, embankment, or pavement soils result in serious and costly damage to structures above them. It is therefore important to estimate the swell potential of these soils. The one dimensional swell potential test is used to estimate the percent swell and swelling pressures developed by the swelling soils. This test can be performed on undisturbed, remolded, or compacted specimens. If the soil structure is not confined (i.e. bridge abutment) such that swelling may occur laterally and vertically, triaxial tests can be used to determine three dimensional swell characteristics.

Chemical (corrosivity) test

Prupose and summerized test proceedures of corrosivity of soils

Lab. Test	prupose	summarized Test procedures and Standards	Remarks
Corrosivity of Soils	To determine the aggressiveness and corrosivity of soils, pH, sulfate and chloride content of soils.	Usually the pH of a soil material is determined electrometically by a pH meter which is a potentiometer equipped with a glass-calomel electrode system calibrated with buffers of known pH. Measurements are commonly performed on a suspension of soil, water and/or alkaline (usually calcium chloride) solutions. **Standards:** AASHTO T 288, T 289, T 290, T 291 G 51, D 512, D 1125, D 2976. D 4230 , D 4972	Because of their environment or composition soils may have varying degrees of acidity or alkalinity, as measured by the pH test. Measurements of pH are particularly important for determining corrosion potential where metal piles, culverts, anchors, metal strips, or pipes are to be used. pH is also an important parameter for evaluating the durability of geosynthetics

Prupose and summerized test proceedures of resistivity

Lab. Test	prupose	summarized Test procedures and Standards	Remarks
Resistivity	To determine the corrosion potential of soils.	The laboratory test for measuring the resistivity of soils is performed using dried prepared soil passing the No. 8 screen. The soil is placed in a box approximately 10.2 cm x 15.2 cm x 4.5 cm with electrical terminals attached to the sides of the box such that they remain in contact with the soil. The terminals in turn are connected to an ohmmeter. A reading of the current passing through the dry soil is taken as the baseline reference resistance. The soil material is then removed and 50 ml to 100 ml of distilled water is added and thoroughly mixed, and placed back in the box. Another reading is taken. The conductivity (conductivity is the reverse of resistivity) of the soil as read by the ohmmeter increases as water is added. The procedure is repeated until the conductivity begins dropping. The highest conductivity, or the lowest resistivity, is used to compute the resistivity of the soil. The method is very sensitive to the distribution of water in the soils placed in the box. The resistivity may also vary significantly with the presence of soluble salts in soils. **Standards:** AASHTO T 288, G 57	Where construction materials susceptible to corrosion are to be used in subgrades it is necessary to determine the corrosion potential of soils. This test is routinely performed for structures where metallic reinforcements, soil anchors, nails, culverts, pipes, or piles are included.

Prupose and summerized test proceedures of collapse potential of soils

Lab. Test	Prupose	summarized Test procedures and Standards	Remarks
Collapse Potential of Soils	To estimate the collapse potential of soils	The collapse potential of suspected soils is determined by placing an undisturbed, compacted or remolded specimen in the consolidometer ring and in a loading device at their natural moisture content. A load is applied and the soil is saturated to measure the magnitude of the vertical displacement. **.Standards:** ASTM D5333	Loess or loess type soils is predominantly composed of silts, and contain 3% to 5% clay. Loess deposits are wind blown formations. Loess type deposits have similar composition and they are formed as a result of the removal of organics by decomposition or the leaching of certain minerals (calcium carbonate). In both cases disturbed samples obtained from these deposits will be classified as silt. When dry or at low moisture content the in situ material gives the appearance of a stable silt deposit. At high moisture contents these soils collapse and undergo sudden changes in volume. Loess, unlike other non-cohesive soils, will stand on almost a vertical slope until saturated. It has a low relative density, a low unit weight and a high void ratio. Structures founded on such soils, upon saturation, may be seriously damaged from the collapse of the foundation soils. The collapse during wetting occurs due to the destruction of clay binding which provide the original strength of these soils. It is conceivable that remolding and compacting may also destroy the original structure.

Rock StrengthTests

The laboratory determination of intactrock strength is accomplished by the following tests: point load index, unconfined compression, triaxial compression, Braziliantest, and directshear. The uniaxial (or unconfined) compression test provides the general reference value, having a respective analogy with standard tests on concrete cylinders. The uniaxial compressive strength($q_u = F_u$) is obtained by compressing a trimmed cylindrical specimen in the longitudinal direction and taking the maximum measured force divided by the cross-sectional area.The point load index serves as a surrogate for the UCS and is a simpler test in that irregular pieces of rock core can be used. A direct tensile test requires special end preparation that is difficult for most commercial labs; therefore tensile strength is more often evaluated by compression loading of cylindrical specimens across their diameter (known as the Brazilian test). Direct shear tests are used to investigate frictional characteristics along rock discontinuity features.

Prupose and summerized test proceedures of testing point load test

Lab. Test	Prupose	summarized Test procedures and Standards	Remarks
Point Load Index (Strength)	To determine strength classification of rock materials through an index test.	Rock specimens in the form of core (diametral and axial), cut blocks or irregular lumps are broken by application of concentrated load through a pair of spherically truncated, conical platens. The distance between specimen-platen contact points is recorded. The load is steadily increased, and the failure load is recorded. There is little sample preparation. However, specimens should conform to the size and shape requirements as specified by ASTM. In general, for the diametral test, core specimens with a length-to-diameter ratio of 1.0 are adequate while for the axial test core specimens with length-to-diameter ratio of 0.3 to 1.0 are suitable. Specimens for the block and the irregular lump test should have a length of 50±35 mm and a depth/width ratio between 0.3 and 1.0 (preferably close to 1.0). The test specimens are typically tested at their natural water content. Size corrections are applied to obtain the point load strength index, Is(50), of a rock specimen. A strength anisotropy index, Ia(50), is determined when Is(50) values are measured perpendicular and parallel to planes of weakness. .**Standards:** D 5731	The test can be performed in the field with portable equipment or in the laboratory. The point load index is used to evaluate the uniaxial compressive strength (Fu). On the average, Fu 25 Is (50). However, the coefficient term can vary from 15 to 50 depending upon the specific rock formation, especially for anisotropic rocks. The test should not be used for weak rocks where Fu < 25 MPa.

Prupose and summerized test proceedures of uniaxial compression strength test

Lab. Test	Prupose	summarized Test procedures and Standards	Remarks
Uniaxial Compression Strength Test	To determine the uniaxial compressive strength of rock (qu = Fu = FC).	In this test, cylindrical rock specimens are tested in compression without lateral confinement. The test procedure is similar to the unconfined compression test for soils and concrete. The test specimen should be a rock cylinder of length-to-width ratio (H/D) in the range of 2 to 2.5 with flat, smooth, and parallel ends cut perpendicular to the cylinder axis. Originally, specimen diameters of NX size were used (D = 2c in. = 44 mm), yet now the standard size is NQ core (D = 1f in. = 47.6 mm). .**Standards:** ASTM D2938	The uniaxial compression test is most direct means of determining rock strength. The results are influenced by the moisture content of the specimens, and thus should be noted. The rate of loading and the condition of the two ends of the rock will also affect the final results. Ends should be planar and parallel per ASTM D 4543. The rate of loading should be constant as per the ASTM test procedure. Inclined fissures, intrusions, and other anomalies will often cause premature failures on those planes. These should be noted so that, where appropriate, other tests such as triaxial or direct shear tests can be required.

Based on the Laboratory UCS strength test result the intact rocks class

Strength class	UCS (Mpa)	Strength class	UCS (Mpa)
Extremely weak rock	0.25-1.0	Strong rock	50.0-100
Very weak rock	1.0-5.0	Very strong rock	100-250
Weak rock	5.0-25.0	Extremely strong rock	>230
Medium strong rock	25.0-50.0		

Prupose and summerized test proceedures of Brazilian test on intact rocks

Lab. Test	Prupose	summarized Test procedures and Standards	Remarks
Tensile (Brazilian) Test for Intact Rocks	To evaluate the (indirect) tensile shear of intact rock core, FT.	Core specimens with length-to-diameter ratios (L/D) of between 2 to 2.5 are placed in a compression loading machine with the load platens situated diametrically across the specimen. The maximum load (P) to fracture the specimen is recorded and used to calculate the split tensile strength. **Standards:** ASTM D3967	The Brazilian or split-tensile strength (FT) is significantly more convenient and practicable for routine measurements than the direct tensile strength test (T0). The test gives very similar results to those from direct tension (Jaeger & Cook, 1976). It is a more fundamental strength measurement of the rock material, as this corresponds to a more likely failure mode in many situations than compression. Also, note that the point load index is actually a type of Brazilian tensile strength that is correlated back to compressive strength

Prupose and summerized test proceedures of direct shear strength of rock alongplane of weakness

Lab. Test	Prupose	summarized Test procedures and Standards	Remarks
Direct Shear Strength of Rock	To determine the shear strength characteristics of rock along a plane of weakness.	The specimen is placed in the lower half of the shear box and encapsulated in either synthetic resin or mortar. The specimen must be positioned so that the line of action of the shear force lies in the plane of the discontinuity to be investigated, and the normal force acts perpendicular to this surface. Once the encapsulating material has hardened, the specimen is mounted in the upper half of the shear box in the same manner. A strip approximately 5 mm wide above and below the shear surface must be kept free of encapsulating material. The test is then carried out by applying a horizontal shear force T under a constant normal load, N. **Standards:** ASTM D3967	Determination of shear strength of rock specimens is an important aspect in the design of structures such as rock slopes, foundations and other purposes. Pervasive discontinuities (joints, bedding planes, shear zones, fault zones, schistosity) in a rock mass, and genesis, crystallography, texture, fabric, and other factors can cause the rock mass to behave as an anisotropic and heterogeneous discontinuum. Therefore, the precise prediction of rock mass behavior is difficult. For nonplanar joints or discontinuities, shear strength is derived from a combination base material friction and overriding of asperities (dilatancy), shearing or breaking of the asperities, rotations at or wedging of the asperities (Patton, 1966). Sliding on and shearing of the asperities can occur simultaneously. When the normal force is not sufficient to restrain dilation, the shear mechanism consists of the overriding of the asperities. When the normal load is large enough to completely restrain dilation, the shear mechanism consists of the shearing off of the asperities.

Durability

The evaluation of rock durability becomes an issue when the materials are to be subjected to the natural elements, seasonal weather, and repeated cyclesof temperature (e.g., flowing water, wetting and drying, wave action, freeze and thaw, etc.) in its proposeduse.Tests to measure durability depend on the type of rock, on its use in construction, and on the elements to which the rock will be subjected. The bases for durability tests are empirical and the results produced are an indication of the rock's resistance to natural processes; the rock's behavior in actual use may vary greatly from the test results.These tests, however, provide reasonably reliable tools for quality control. The suitability of various types of rock for different uses should, in addition to these test results, depend on their performance in previous applications. An example of the use of rock durability tests is in the evaluation of shale in rockfill embankments.

Prupose and summerized test proceedures of slake durability

Lab. Test	Propose	summarized Test procedures and Standards	Remarks
Slake Durability	To determine the durability of shale or other weak or soft rocks subjected to cycles of wetting and drying.	In this test dried fragments of rock of known weight are placed in a drum fabricated with 2.0 mm square mesh wire cloth. The drum is rotated in a horizontal position along its longitudinal axis while partially submerged in distilled water to promote wetting of the sample. The specimens and the drum are dried at the end of the rotation cycle (10 minutes at 20 rpm) and weighed. After two cycles of rotating and drying the weight loss and the shape and size of the remaining rock fragments are recorded and the Slake Durability Index (SDI) is calculated. Both the SDI and the description of the shape and size of the remaining particles are used to determine the durability of soft rocks. **Standards:** ASTM D4644	This test is typically performed on shales and other weak rocks that may be subject to degradation in the service environment. When some shals are newly exposed to atmospheric conditions, they can degrade rapidly and affect the stability of a rock fill or cut, the subgrade on which a foundation is to be placed, or the base and side walls of drilled shafts prior to placement of concrete.

Propose and summerized test proceedures of soundness of riprap

Lab. Test	Prupose	summarized Test procedures and Standards	Remarks
Soundness of Riprap	To determine the soundness of rock subjected to erosion.	The procedure is known as the Rock Slab Soundness Test. Two representative, sawed, rock slab specimens are immersed in a solution of sodium or magnesium sulfate and dried and weighed for five cycles. The percent weight loss as a result of these tests is expressed as percent soundness. **Standards:** ASTM D5240	One of the most effective means to control erosion along riverbanks and coastal beaches is by covering exposed soil with rip-rap, or a combination of geosynthetics and rip-rap. Rock or stone used in this mode is subject to degradation from weathering effects due to repeated cycles of wetting & drying, as well as repeated exposure to salts used in de-icing of roadways. This test is used to estimate this type of degradation. A similar test for aggregates is available through ASTM C 88.

Deformation Characteristics of Intact Rocks

The stiffness of rocks is represented by an equivalent elastic modulus at small-to intermediate-strains.

Prupose and summerized test proceedures of elastic moduli

Lab. Test	Prupose	summarized Test procedures and Standards	Remarks
Elastic Moduli	To determine the deformation characteristics of intact rock at intermediate strains and permit comparison with other intact rock types.	This test is performed by placing an intact rock specimen in a loading device and recording the deformation of the specimen under axial stress. The Young's modulus, average, secant, or tangent moduli, can be determined by plotting axial stress versus axial strain curves. **Standards:** ASTM D5240	The results of these tests cannot always be replicated because of localized variations in the each unique rock specimen. They provide reasonably reliable data for engineering applications involving rock classification type, but must be adjusted to take into account rock mass characteristics such as jointing, fissuring, and weathering.

Prupose and summerized test proceedures of ultrasonic testing

Lab. Test	Prupose	summarized Test procedures and Standards	Remarks
Ultrasonic Testing	To determine the pulse velocities of compression and shear waves in intact rock and the ultrasonic elastic constants of isotropic rock.	Ultrasound waves are transmitted through a carefully prepared rock specimen. The ultrasonic elastic constants are calculated from the measured travel time and distance of compression and shear waves in a rock specimen. Figure 8-7 shows a schematic diagram of typical apparatus used for ultrasonic testing. **Standards:** ASTM D2845	The primary advantages of ultrasonic testing are that it yields compression (P-wave) and shear (S-wave) velocities, and ultrasonic values for the elastic constants of intact homogeneous isotropic rock specimens. Elastic constants for rocks having pronounced anisotropy may require measurements to be taken across different directions to reflect orthorhombic stiffnesses and moduli, particularly if pronounced foliation, banding, layering, and fabric are evident.

APPENDIX II: Seismic hazard assessment and study

Seismic Intensity and magnitude

General

Although earthquakes have been reported from all parts of the world they are primarily associated with areas of recent mountain building and with the global rift system, in other words with the edge of the plates which form the earth's crust. The earth's crust is being slowly displaced at the margins of the plates, presumably by convection currents in the upper mantle. Differential displacements give rise to elastic strains, which eventually exceed the strength of the rocks involved and faults then occur. The strained rocks rebound along the fault under the elastic stresses until the strain is partly or wholly dissipated.

Intensity and magnitude of earthquakes

In earthquake regions it is necessary for the engineer to establish the nature of the risk to a new structure. Thus he has to assess the probability of the occurrence of earthquakes, their intensity and magnitude, and the likelihood of earthquake damage to structure concerned. As soon as the intensity detrimental to the planned structure is established, the probability of an earthquake of this intensity in the given region should be estimated.

Intensity

Earthquake intensity scales depend on human perceptibility and the destructivity of earthquake. Whereas the degree of damage may be estimated correctly and objectively, the perceptibility of an earthquake depends on the location of the observer and his sensibility.

Magnitude

As noted above the intensity of an earthquake is characterized by its effects and is a qualitative concept, whereas its magnitude is an instrumentally measured quantity related to the total energy released during an earthquake. The magnitude assigned to a given earthquake therefore corresponds to the highest intensity of that earthquake.

Methods of seismic investigation

There is no method at present of forecasting the exact location, size or time of an earthquake. However, it can be assumed that a probable prediction is reasonable and that past patterns of seismic activity will continue. Hence earthquake risk reports should take into account an appraisal of known faults, the distance from major faults, the number of recorded earthquakes, the history of damage, and an estimate of magnitude and intensity of the strongest shock expected. The latter must take the ground conditions at the site into consideration.

Many text books handle the problem of earthquakes and civil engineering exclusively from the viewpoint of what is likely to happen to a structure when subjected to earthquake vibration; topics such as the point of origin of earthquake vibrations, the frequency of earthquake events etc. are not considered. Similarly some earthquake seismologists, when considering the behavior of the ground under the earth tremors, seem to display a lack of soil mechanics. Proper handling of an earthquake problem requires the following components of knowledge:

1. An estimate of the likely strength, frequency and location of future earthquakes. This may be assessed by a study of the geology of the region around the construction site and a survey of the past earthquake events.
2. A study of site geology in order to assess the likely ground response to a possible future earthquake event. This would determine whether any possibilities existed of phenomena such as liquefactions, land spreading, flow slides etc, which, are associated with soft saturated Quaternary deposits.
3. An assessment of the likely response of the proposed structure to the anticipated tremors and any other ground response events associated with the earthquake.

Seismic zoning and data output to the intended design
Maps can be drawn, by using seismic evidence and historical data, which indicate the epicenter areas of earthquakes and these are then zoned according to activity. The longer the history on which these maps are based, the better they are likely to be. A seismic zoning map therefore, shows the zones of different seismic danger in a particular area. Hence it provides the engineer with a broad picture of the earthquake risk that can be involved in seismic regions.

Consequently one of the tasks of earthquake engineers is to prepare detailed seismic zoning maps which take account of local engineering and geological characteristics as well as the differences in the spectrum of seismic vibrations and, most important of all, the probability of the occurrence of earthquakes of various intensities.

The East African Rift System is considered by many to be the best example of continental rifting on the land. It represents the most extensive currently active zone of continental rifting on earth. The distribution of earthquake epicenters in Ethiopia is found to be associated with seismically active faults. According to the Ethiopian building Codes Standard (ES EN 1998 – 1: 2015), the country has been subdivided into six seismic zones see figure9. 1, the hazard within each zone can be assumed to be constant. There are six zones ranging from 0-5, 0 being no seismic hazard while 5 being the highest seismic zone which is located within rift valley of Ethiopia. The design ground acceleration, chosen in figure 9.1 for each seismic zone, corresponds to a reference return period of 475 years (10% probability of exceedance in 50 years). To this reference return period, an importance factor I equal to 1.0 is assigned.

According to Ethiopian Building Code Standard, the following bedrock accelerations ratio is applied to seismic zones shown in figure 1.

Bedrock acceleration ratio α_o

Zone	5	4	3	2	1	0
$\alpha_o = a_g/g$	0.20	0.15	0.10	0.07	0.04	0

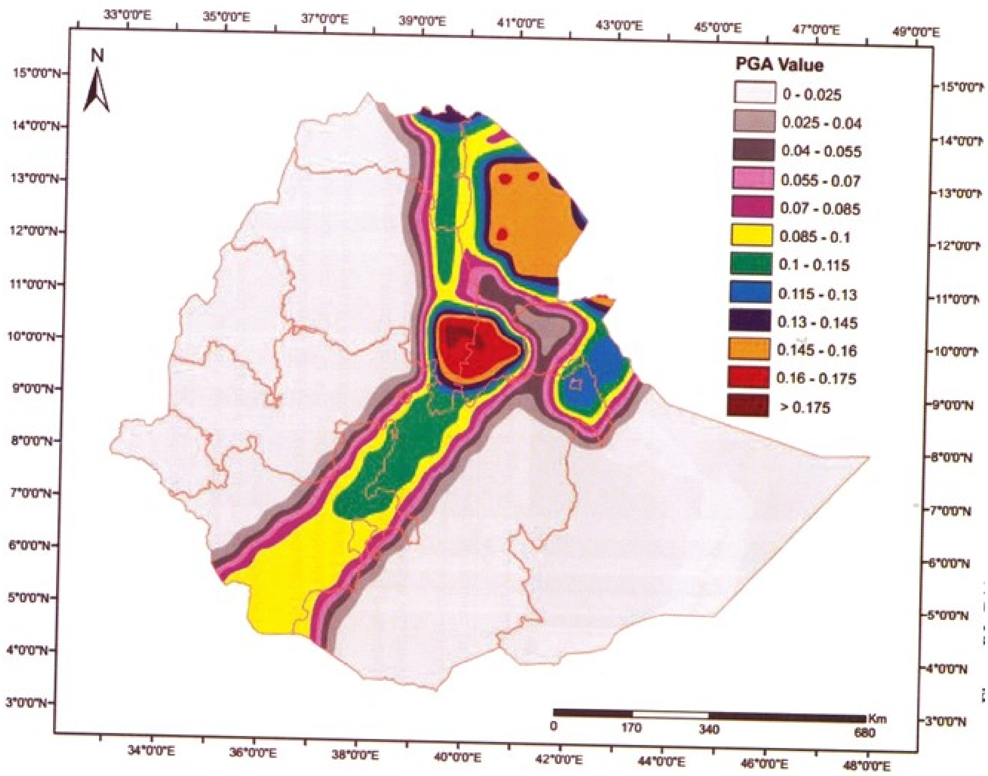


Figure 1: Pick ground acceleration zonation map of Ethiopia

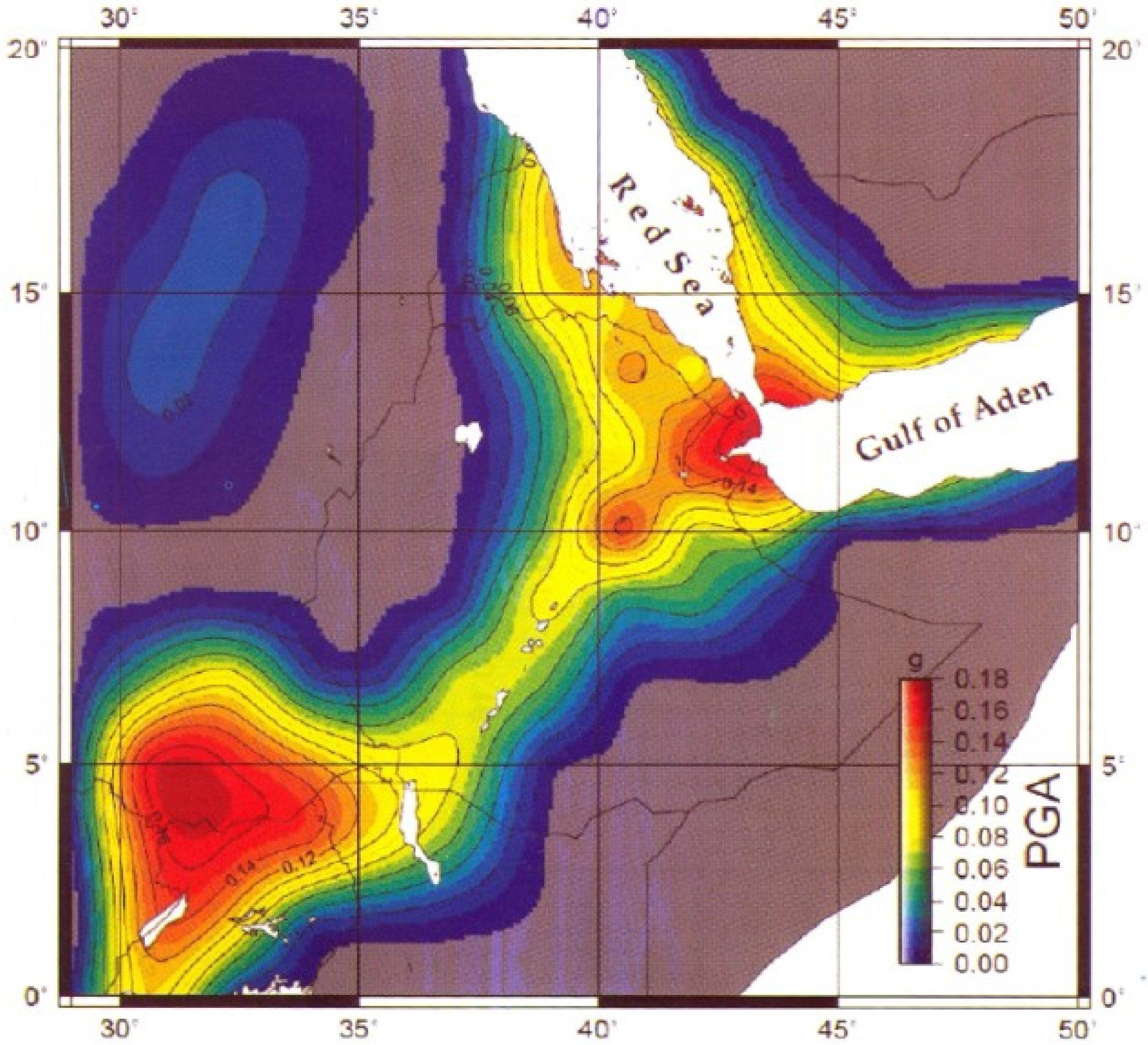
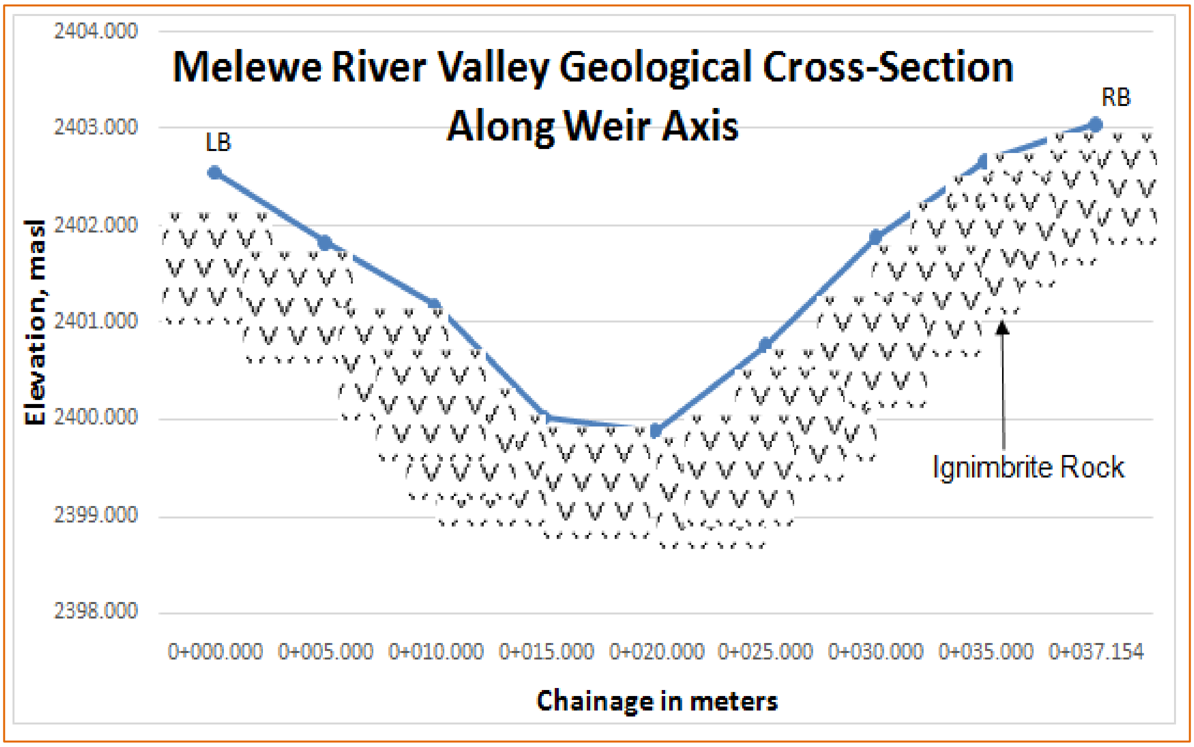
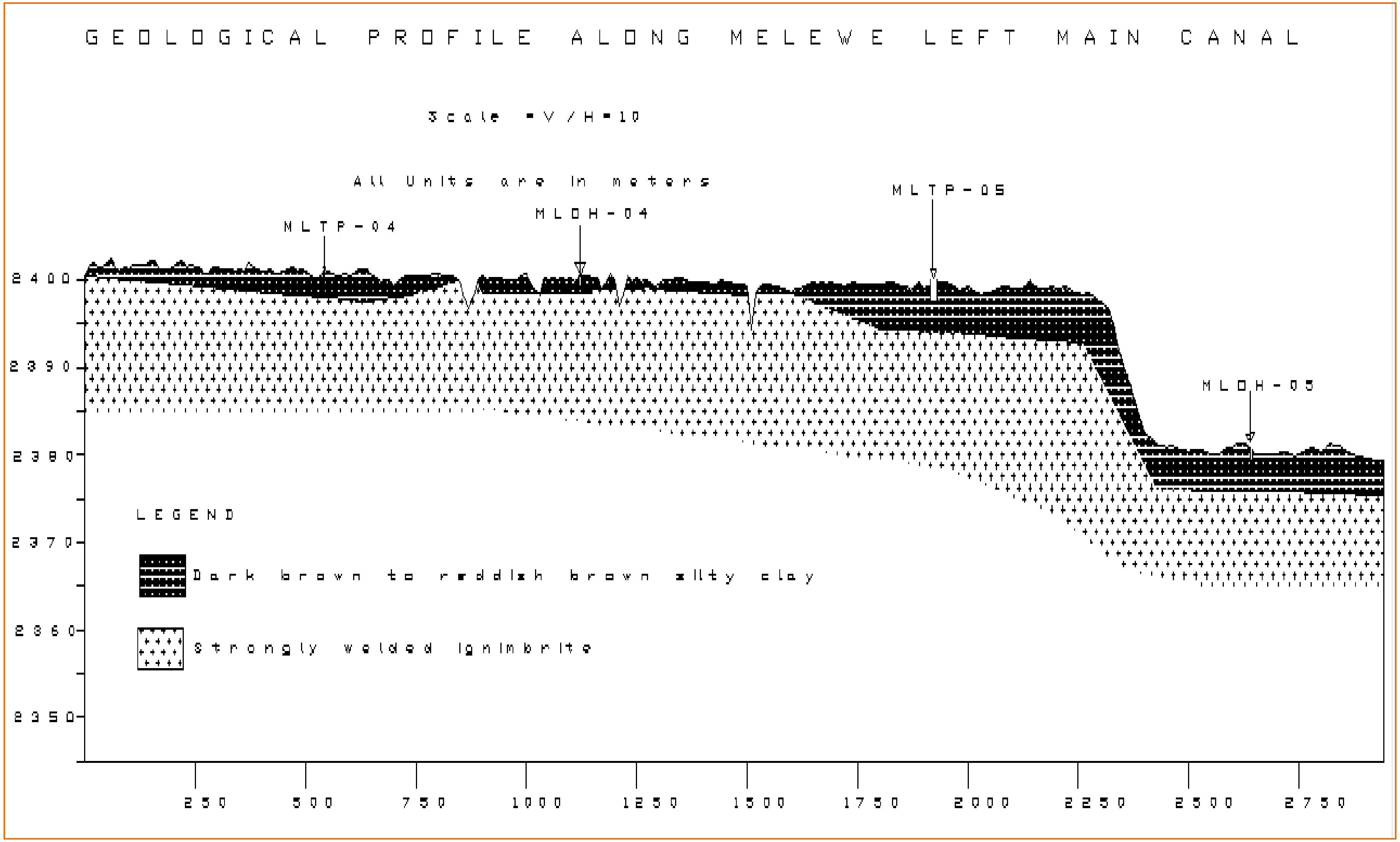
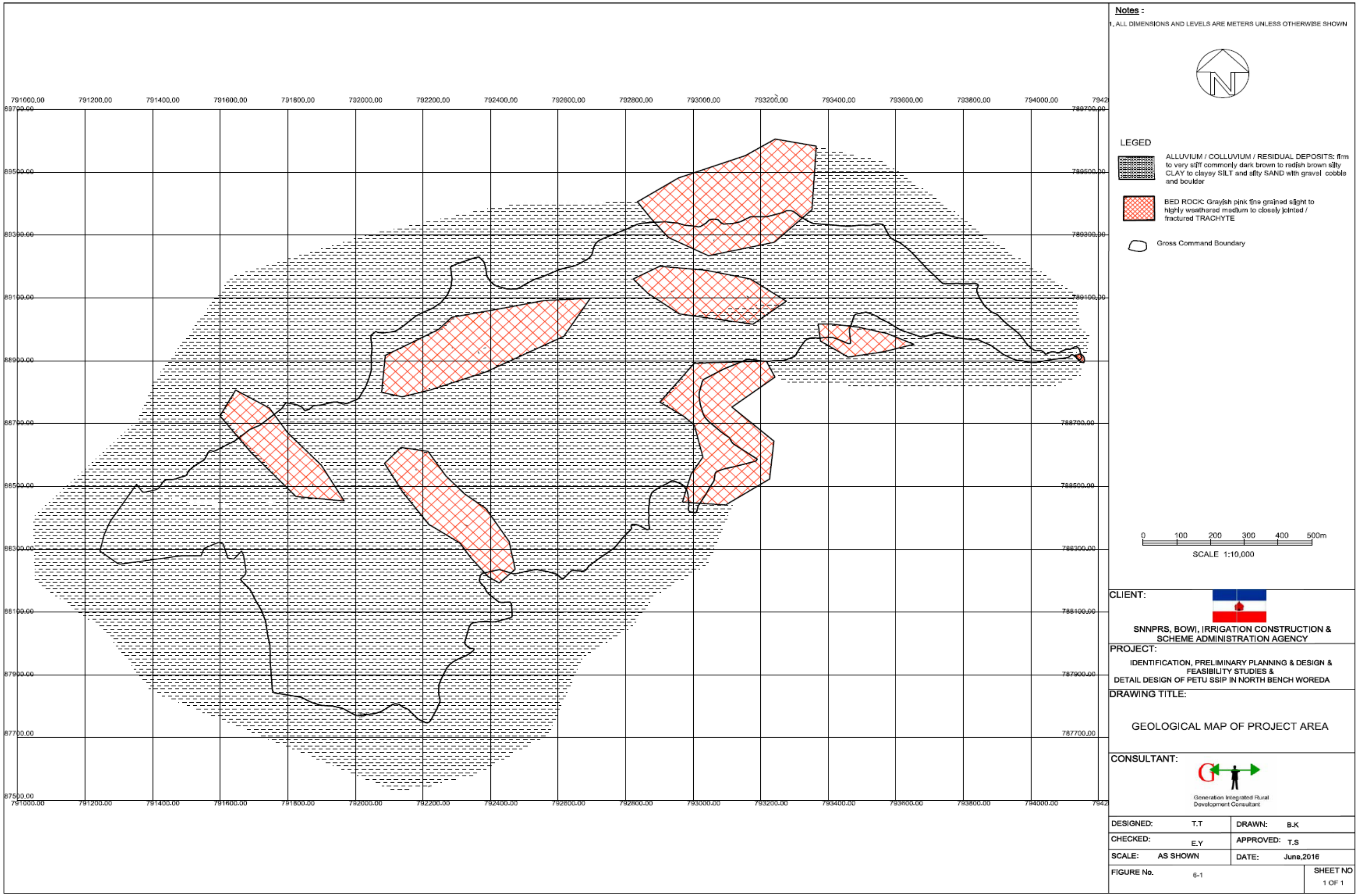


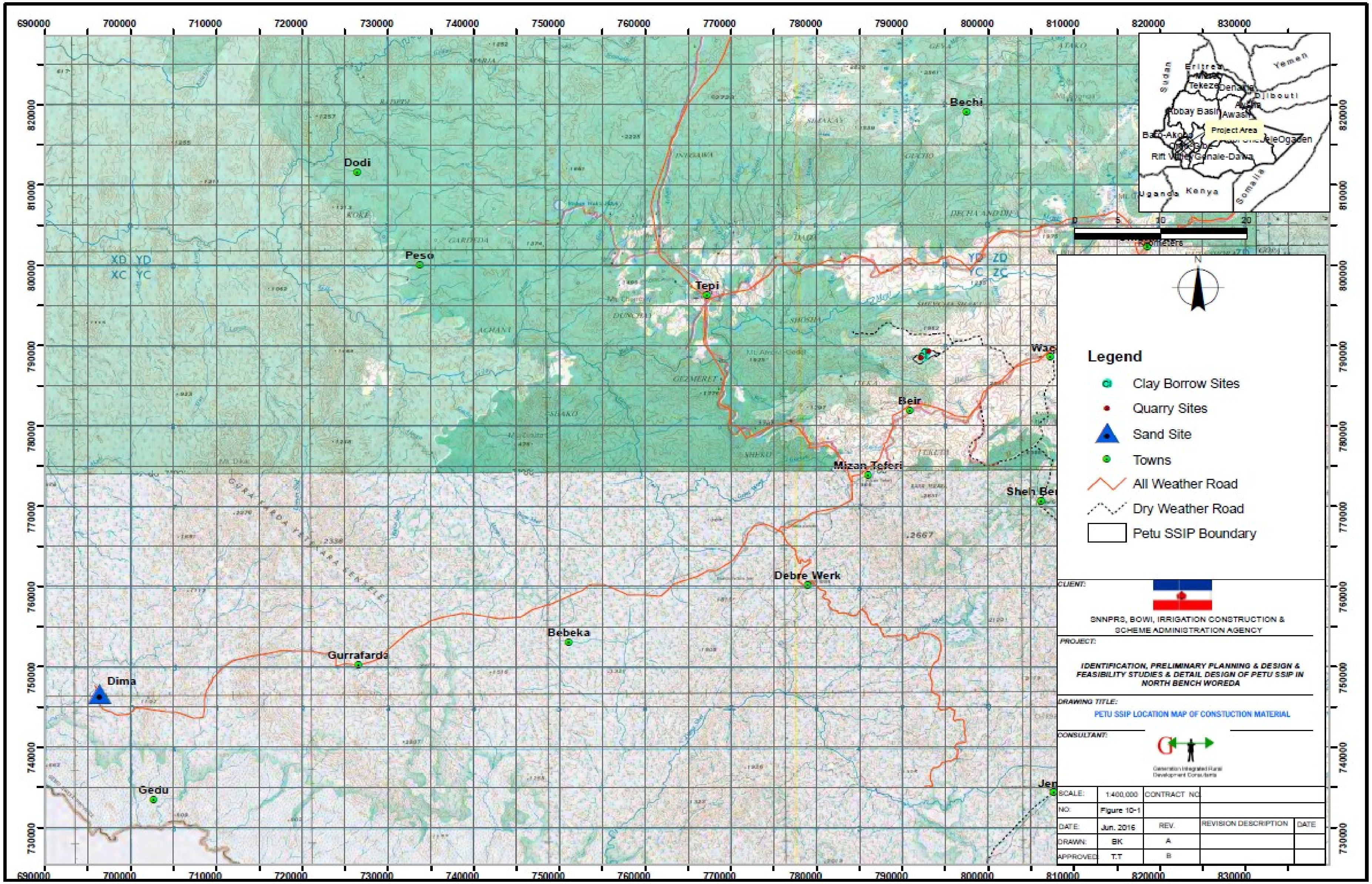
Figure 2: Seismic hazard map along the horn of Africa (ES EN 1998 – 1: 2015)

APPENDIX III: Sample geological X-Section, geological maps….etc produced on different projects

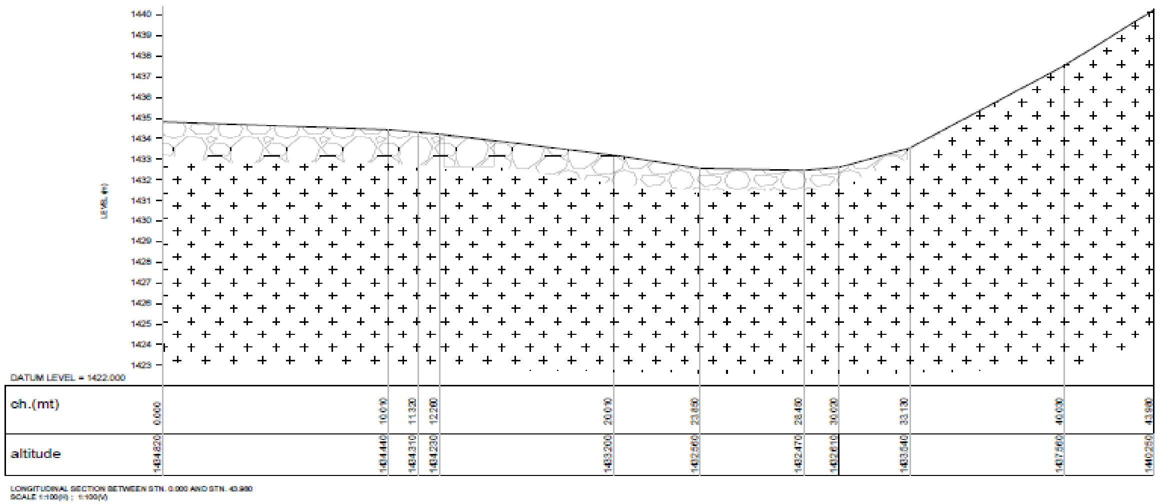



GEOLOGICAL PROFILE ALONG MELEWE LEFT MAIN CANAL
Scale =V/H=10
All Units are in meters
MLTP-04
MLOH-04
MLTP-05
MLOH-05
2400
2390
2380
2370
2360
2350
250
500
750
1000
1250
1500
1750
2000
2250
2500
2750
LEGEND
Dark brown to reddish brown silty clay
Strongly welded ignimbrite


Notes :
1. ALL DIMENSIONS AND LEVELS ARE METERS UNLESS OTHERWISE SHOWN
N
LEGED
ALLUVIUM / COLLUVIUM / RESIDUAL DEPOSITS: firm to very stiff commonly dark brown to redish brown silty CLAY to clayey SILT and silty SAND with gravel cobble and boulder
BED ROCK: Grayish pink fine grained slight to highly weathered medium to closely jointed / fractured TRACHYTE
Gross Command Boundary
0 100 200 300 400 500m
SCALE 1:10,000
CLIENT:
SNNPRS, BOWI, IRRIGATION CONSTRUCTION & SCHEME ADMINISTRATION AGENCY
PROJECT:
IDENTIFICATION, PRELIMINARY PLANNING & DESIGN & FEASIBILITY STUDIES &
DETAIL DESIGN OF PETU SSIP IN NORTH BENCH WOREDA
DRAWING TITLE:
GEOLOGICAL MAP OF PROJECT AREA
CONSULTANT:
Generation Integrated Rural Development Consultant
DESIGNED: T.T
DRAWN: B.K
CHECKED: E.Y
APPROVED: T.S
SCALE: AS SHOWN
DATE: June,2016
FIGURE No. 6-1
SHEET NO 1 OF 1



Bechi
Dodi
Peso
Tepi
Beir
Mizan Teferi
Debre Werk
Bebeka
Gurrafarda
Dima
Gedu
XD YD
XC YC
YD ZD
YC ZC
Eritrea
Sudan
Yemen
Djibouti
Tekeze
Abbay Basin
Awash
Project Area
Ogaden
Rift Valley
Genale-Dawa
Uganda
Kenya
Somalia
Kilometers
Legend
Clay Borrow Sites
Quarry Sites
Sand Site
Towns
All Weather Road
Dry Weather Road
Petu SSIP Boundary
CLIENT:
SNNPRS, BOWI, IRRIGATION CONSTRUCTION & SCHEME ADMINISTRATION AGENCY
PROJECT:
IDENTIFICATION, PRELIMINARY PLANNING & DESIGN & FEASIBILITY STUDIES & DETAIL DESIGN OF PETU SSIP IN NORTH BENCH WOREDA
DRAWING TITLE:
PETU SSIP LOCATION MAP OF CONSTUCTION MATERIAL
CONSULTANT:
Generation Integrated Rural Development Consultants
SCALE: 1:400,000
CONTRACT NO.
NO: Figure 10-1
DATE: Jun. 2016
REV.
REVISION DESCRIPTION
DATE
DRAWN: BK
A
APPROVED: T.T
B



Notes :

1. ALL DIMENSIONS AND LEVELS ARE METERS UNLESS OTHERWISE SHOWN

LEGEND

ALLUVIUM: LOOSE TO STIFF VARYGATED DARK TO DARK BROWN GRAVEL SILT CLAY WITH BOULDER AND COBBLE ROCK FRAGMENTS

BED ROCK: GRAYISH PINK FINE GRAINED SLIGHT TO MODERATELY WEATHERED WIDE TO CLOSELY JOINTED TRACHYTE

0 20 40 60 80 100m
SCALE 1:2000

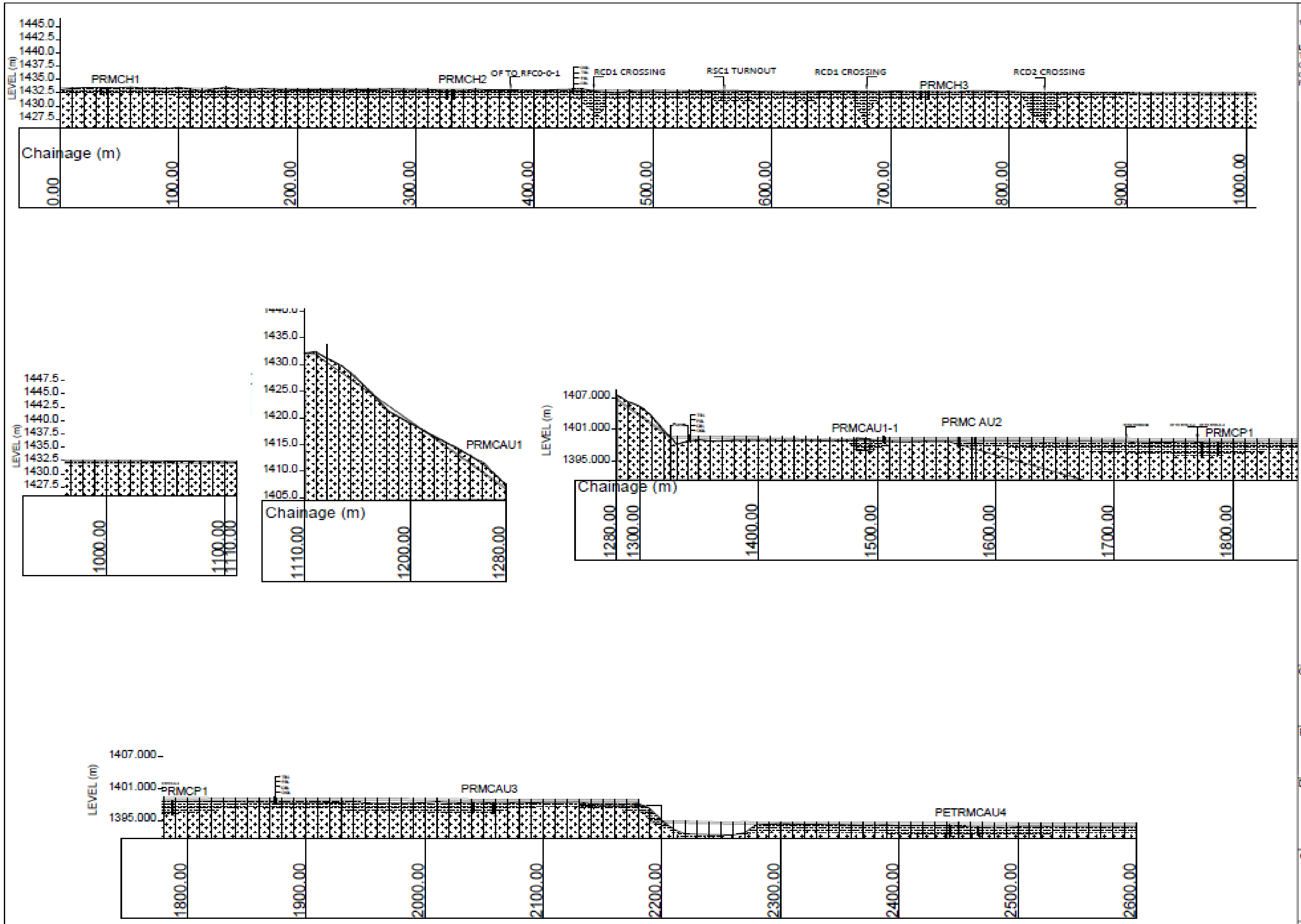
CLIENT:
SNNPRS, BOWI, IRRIGATION CONSTRUCTION & SCHEME ADMINISTRATION AGENCY

PROJECT:
IDENTIFICATION, PRELIMINARY PLANNING & DESIGN & FEASIBILITY STUDIES & DETAIL DESIGN OF PETU SSIP IN NORTH BENCH WOREDA

DRAWING TITLE:
GEOLOGICAL CROSS - SECTION ALONG THE PROPOSED HEADWORK SITE

CONSULTANT:
Generation Integrated Rural Development Consultant

DESIGNED:	T.T	DRAWN:	B.K
CHECKED:	E.Y	APPROVED:	T.S
SCALE:	AS SHOWN	DATE:	June,2016
FIGURE No.	6-2	SHEET NO	1 OF 1



Notes :
1. ALL DIMENSIONS AND LEVELS ARE METERS UNLESS OTHERWISE SHOWN

LEGEND
TBL = TOP BANK LEVEL
CBL = DESIGN BED LEVEL
OGL = ORIGINAL GROUND LEVEL
FSL = FULL SUPPLY LEVEL

LEGEND

UNCONSOLIDATED SEDIMENTS: STIFF TO VERY STIFF DARK BROWN TO REDISH BROWN AND DARK SILTY CLAY TO CLAYEY SANDY SILT INCLUDING STIFF RESIDUAL SOIL, LOSS TO DENSE COBBLE GRAVEL BOULDER. EXCAVATION USING EXCAVATOR.

BED ROCK: FINE GRAINED MEDIUM TO CLOSELY JOINTED / FRACTURED SLIGHT TO HIGHLY WEATHERED TRACHYTE EXCAVATION USING DOZER, JACK HAMER, LINING RECOMMENDED

PRMC AU6 — EXPLORATION HOLE

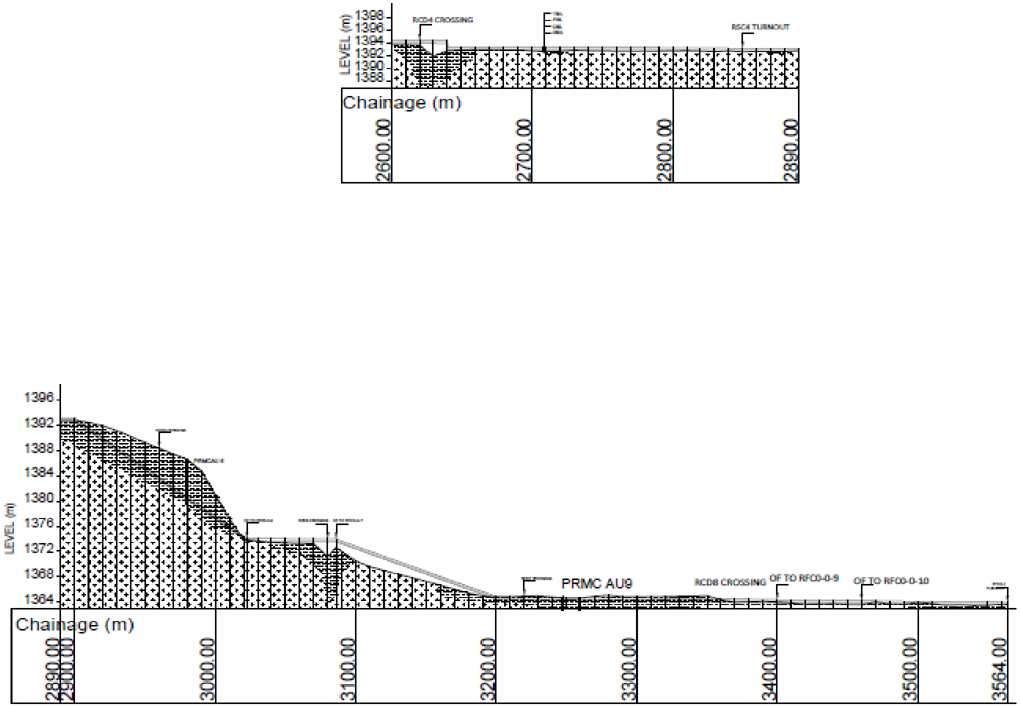
CLIENT:
SNNPRS, BOWI, IRRIGATION CONSTRUCTION & SCHEME ADMINISTRATION AGENCY

PROJECT:
IDENTIFICATION, PRELIMINARY PLANNING & DESIGN & FEASIBILITY STUDIES & DETAIL DESIGN OF PETU SSIP IN NORTH BENCH WOREDA

DRAWING TITLE:
RIGHT MAIN CANAL (RMC) GEOLOGICAL X SECTION

CONSULTANT:
Generation Integrated Rural Development Consultant

DESIGNED:	T.T	DRAWN:	B.K
CHECKED:	E.Y	APPROVED:	T.B
SCALE:	AS SHOWN	DATE:	June,2016
FIGURE No.	6-3	SHEET NO	1 OF2



Notes :

LEGEND

UNCONSOLIDATED SEDIMENTS: STIFF TO VERY STIFF DARK BROWN TO REDISH BROWN AND DARK SILTY CLAY TO CLAYEY SANDY SILT INCLUDING STIFF RESIDUAL SOIL, LOSS TO DENSE COBBLE GRAVEL BOULDER.
EXCAVATION USING EXCAVATOR.

BED ROCK: FINE GRAINED MEDIUM TO CLOSELY JOINTED / FRACTURED SLIGHT TO HIGHLY WEATHERED TRACHYTE EXCAVATION USING DOZER, JACK HAMER, LINING RECOMMENDED

PRMC AU6
EXPLORATION HOLE

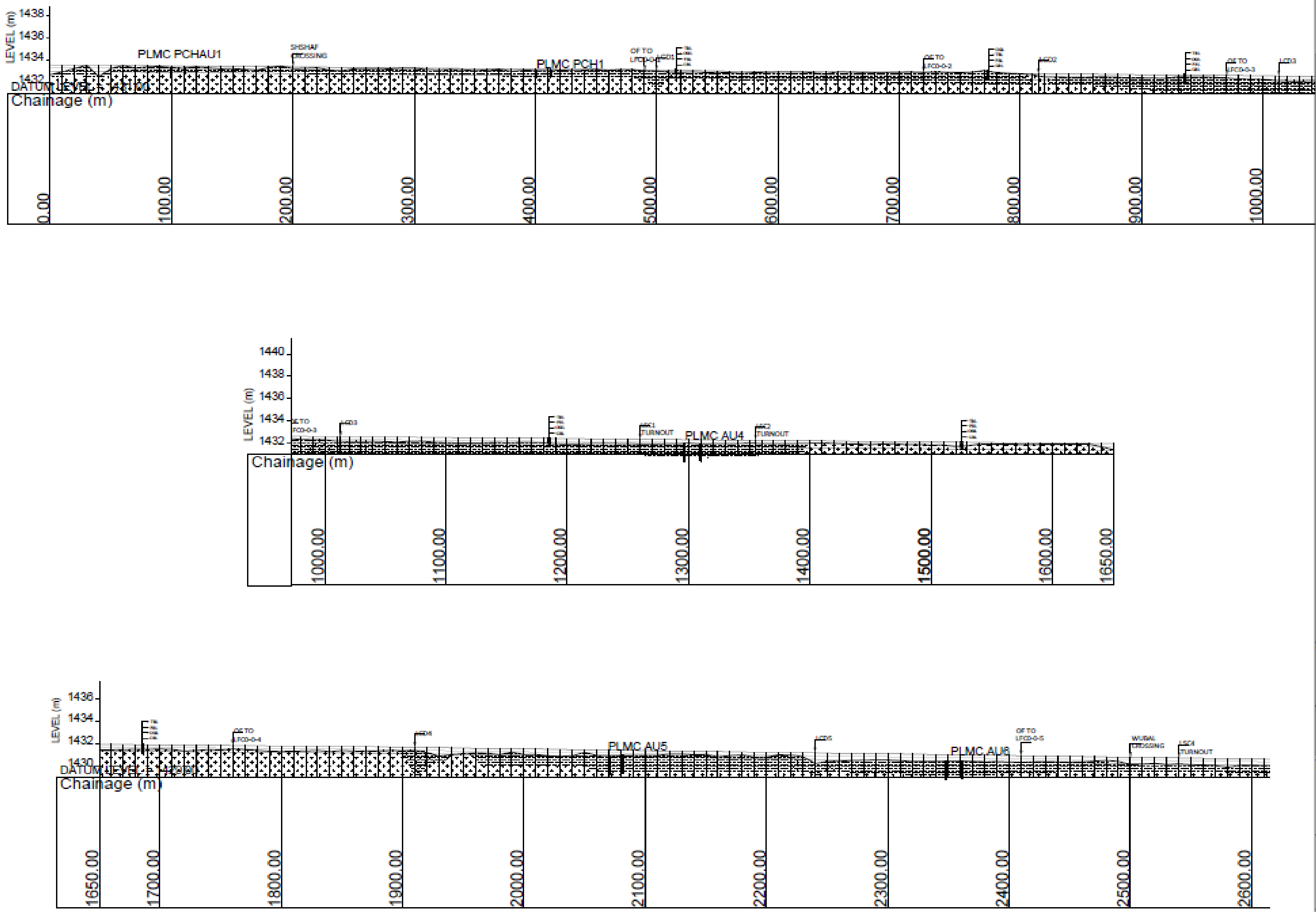
CLIENT:
SNNPRS, BOWI, IRRIGATION CONSTRUCTION & SCHEME ADMINISTRATION AGENCY

PROJECT:
IDENTIFICATION, PRELIMINARY PLANNING & DESIGN & FEASIBILITY STUDIES & DETAIL DESIGN OF PETU SSIP IN NORTH BENCH WOREDA

DRAWING TITLE:
RIGHT MAIN CANAL (RMC)
GEOLOGICAL X SECTION

CONSULTANT:
Generation Integrated Rural Development Consultant

DESIGNED:	T.T	DRAWN:	B.K
CHECKED:	E.Y	APPROVED:	T.S
SCALE:	AS SHOWN	DATE:	June,2016
FIGURE No.	6-3	SHEET NO	2 OF2



Notes :

1. ALL DIMENSIONS AND LEVELS ARE METERS UNLESS OTHERWISE SHOWN.

LEGEND

UNCONSOLIDATED SEDIMENTS: STIFF TO VERY STIFF DARK BROWN TO REDISH BROWN AND DARK SILTY CLAY TO CLAYEY SANDY SILT, RESIDUAL SOIL/ RIVER DEPOSITS LOSS TO DENSE COBBLE GRAVEL BOULDER. EXCAVATION USING EXCAVATOR AND MANUAL LABOR USING HAND TOOLS.

BED ROCK: FINE GRAINED MEDIUM TO CLOSELY JOINTED / FRACTURED SLIGHT TO HIGHLY WEATHERED TRACHYTE EXCAVATION USING MACHINERIES LINING RECCOMENDED

PLMC AU2

EXPLORATION HOLE

CLIENT:

SNNPRS, BOWI, IRRIGATION CONSTRUCTION & SCHEME ADMINISTRATION AGENCY

PROJECT:

IDENTIFICATION, PRELIMINARY PLANNING & DESIGN & FEASIBILITY STUDIES & DETAIL DESIGN OF PETU SSIP IN NORTH BENCH WOREDA

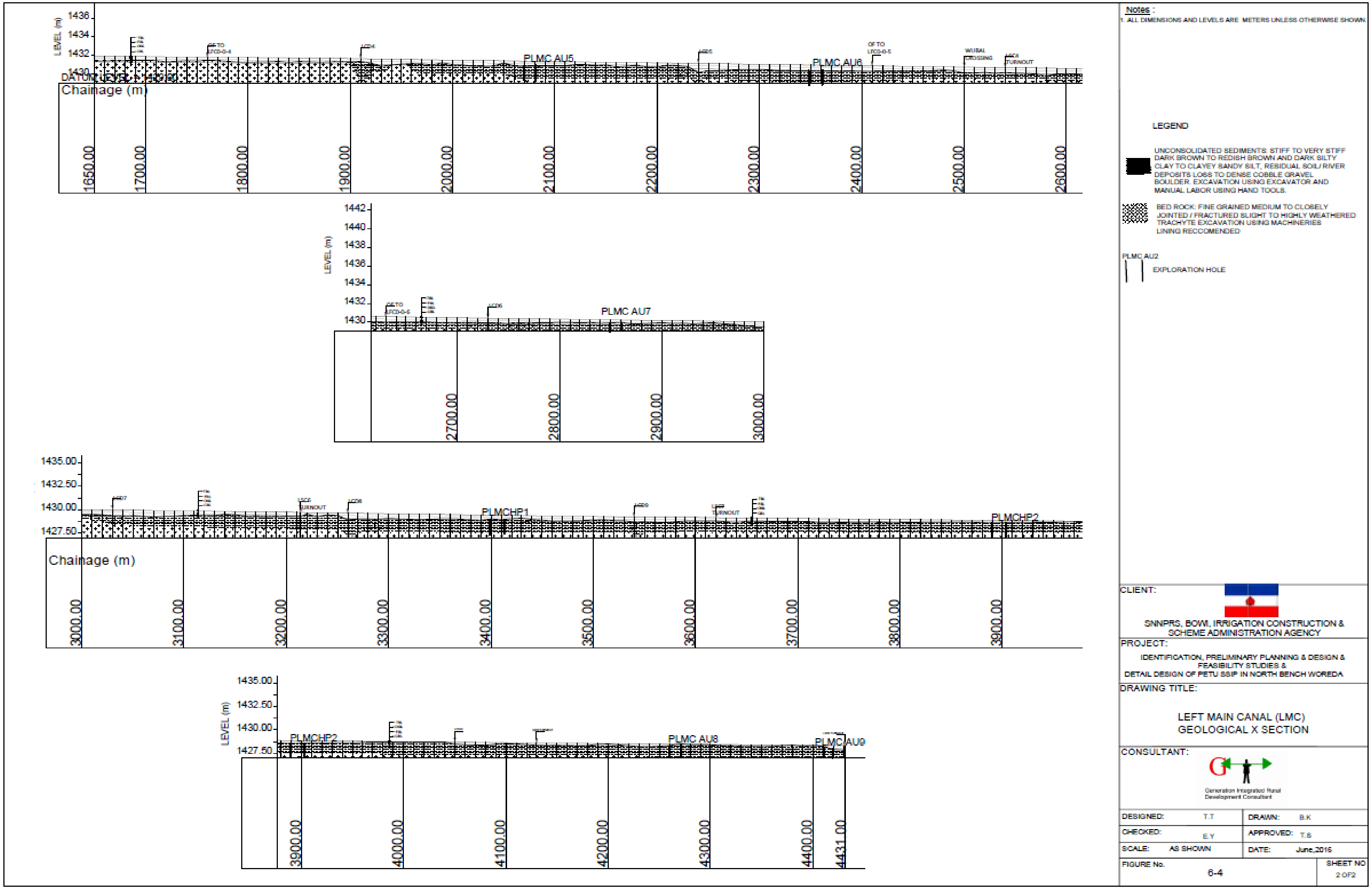
DRAWING TITLE:

LEFT MAIN CANAL (LMC)
GEOLOGICAL X SECTION

CONSULTANT:

Generation Integrated Rural Development Consultant

DESIGNED:	T.T	DRAWN:	B.K
CHECKED:	E.Y	APPROVED:	T.S
SCALE:	AS SHOWN	DATE:	June,2016
FIGURE No.	6-4	SHEET NO	1 OF2


Notes :
1. ALL DIMENSIONS AND LEVELS ARE METERS UNLESS OTHERWISE SHOWN.
LEGEND
UNCONSOLIDATED SEDIMENTS: STIFF TO VERY STIFF DARK BROWN TO REDISH BROWN AND DARK SILTY CLAY TO CLAYEY SANDY SILT, RESIDUAL SOIL/ RIVER DEPOSITS LOSS TO DENSE COBBLE GRAVEL BOULDER. EXCAVATION USING EXCAVATOR AND MANUAL LABOR USING HAND TOOLS.
BED ROCK: FINE GRAINED MEDIUM TO CLOSELY JOINTED / FRACTURED SLIGHT TO HIGHLY WEATHERED TRACHYTE EXCAVATION USING MACHINERIES LINING RECCOMENDED
PLMC AU2
EXPLORATION HOLE
CLIENT:
SNNPRS, BOWI, IRRIGATION CONSTRUCTION & SCHEME ADMINISTRATION AGENCY
PROJECT:
IDENTIFICATION, PRELIMINARY PLANNING & DESIGN & FEASIBILITY STUDIES & DETAIL DESIGN OF PETU SSIP IN NORTH BENCH WOREDA
DRAWING TITLE:
LEFT MAIN CANAL (LMC) GEOLOGICAL X SECTION
CONSULTANT:
DESIGNED: T.T
DRAWN: B.K
CHECKED: E.Y
APPROVED: T.S
SCALE: AS SHOWN
DATE: June,2016
FIGURE No. 6-4
SHEET NO 2 OF2
PLMC AU5
PLMC AU6
PLMC AU7
PLMCHP1
PLMCHP2
PLMC AU8
PLMC AU9
Chainage (m)
LEVEL (m)

SSIGL 6

Prepared by

GIRDC