# GUIDELINES

# FOR

# CONSULTING GEOTECHNICAL

# ASSIGNMENTS

**GEOTECHNICAL SERVICES**
**TECHNICAL STANDARDS BRANCH**
**TRANSPORTATION AND CIVIL ENGINEERING DIVISION**
**MAY 1998 (Revised)**

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# FOREWORD

This document provides information to the geotechnical consultant in the preparation of proposal to allow the execution of an assignment to satisfy the requirements of Alberta Transportation (AT). Such assignments include past AT practice of hiring geotechnical consultants (engineers and technologists) for drilling investigations and instrumentation programs for the Department.

This document is only a general reference (not a design manual or department standard). All Alberta Transportation (AT) current documents may be purchased from AT or viewed and downloaded from the website, www.infras.gov.ab.ca.

Various past practices (in the form of documentation formats, borehole presentation and reporting, and requirements from other departments and regional offices, etc) are included in the Appendices. Some of these may have become out-dated and it is the responsibility of the reader to reference current versions of these documents. One such example is the department's gradual phasing out of the old DOS ESEBase software used in logging borehole information. In order to make the borehole data compatible with the Department's Transportation Infrastructure Management System (TIMS), the gINT Window Software (www.gcagint.com) has been adopted to convert old Department borehole logs to this format, and the consultants will be required to switch to this new format.

There is no warranty, expressed or implied, made on the accuracy of the contents of this document or their extraction from referenced publications. AT assumes no responsibility for errors or omissions or possible misinterpretation that may result from the use of the material herein contained.

Geotechnical Services
Technical Standards Branch
Transportation and Civil Engineering Division
Alberta Transportation
May 1998 (Revised)

# 1 GEOTECHNICAL REPORT CONSIDERATIONS

GENERAL

The geotechnical report is a tool used to communicate the site conditions, and design and construction recommendations. The importance of preparing an adequate geotechnical report cannot be over-stressed. The information contained in this report is often referred during design and construction and frequently after completion of the project (for example, to resolve claims). Therefore, the report must be clear, concise and accurate. Both an adequate site investigation and a comprehensive geotechnical report are necessary to construct a safe and cost-effective project.

The geotechnical report must contain certain basic essential information, including:

- Summary of all subsurface exploration data, including subsurface soil profile, exploration logs, laboratory or in-situ test results, and ground water information;
- Interpretation and analysis of the subsurface data;
- Specific engineering recommendations for design;
- Discussion of conditions which may be encountered during construction, including recommendations for solution of anticipated problems; and
- Recommended geotechnical special provisions for contract documents.

A typical geotechnical report would contain the following chapters:

1. Introduction
   - Terms of reference
   - Background information
   - Scope of works
   - Information from office and field reconnaissance

2. Site Description and Characteristics
   - Site geology
   - Airphoto interpretation
   - Previously provided information
   - Site review

3. Site Investigation/Field Work
   - The equipment and procedure used for drilling testholes
   - The types of soil and rock samples and the procedures used in their collection
   - The field tests and the procedures used
   - Testhole logs

4. Analysis of Field & Laboratory Test Results
   - Stratification profile along important cross-sections
   - Ground water table conditions
   - Demarcation of site into several homogeneous zones and the average engineering properties of soil for each zone

5. Recommendation and Conclusion
   - Recommendations for foundation types
   - Dimension of foundation elements
   - Design details for retaining walls, embankments and slopes
   - Extent of borrow areas where applicable
   - Recommended construction procedures
   - Recommendations for field instrumentation
   - Recommended geotechnical special provisions for contract documents

# 2 SITE INVESTIGATION CONSIDERATIONS

Site investigation is considered the most important step in the geotechnical analysis and design process. Conducting an adequate site investigation, presentation of the subsurface information in the geotechnical report and on the plan deserves careful attention. The following are some of the considerations that apply to this aspect of work and must be given due consideration.

Geotechnical Report

1. Description of the general location of the investigation

2. Summary of scope and purpose of the investigation

3. Concise description of geologic setting and topography of area

4. List of field exploration and laboratory tests on which the report is based

5. General description of subsurface soil, rock and groundwater conditions

6. Information included with the geotechnical report (typically included in report appendices):
   - Testhole logs
   - Field test data
   - Laboratory test data
   - Photographs (if pertinent)

Plan and Subsurface Profile

1. Plan and subsurface profile of the investigation site

2. Site investigation Information

3. Location of field explorations on the plan view

4. Plot of explorations on a profile at their true elevation and location

5. Description and/or graphic description of soil and rock types

6. Groundwater levels and date measured

Subsurface Profile or Field Boring Log

1. Sample types and depths

2. SPT blow counts, percent core recovery and RQD values

Laboratory Test Data

1. Soil classification tests such as natural moisture content, gradation, Atterberg Limits, performed on selected representative samples to verify field visual soil identifications.

2. Laboratory test results such as shear strength consolidation included and/or summarized.

# 3 EMBANKMENT/APPROACH FILL CONSIDERATIONS

## EMBANKMENTS OVER SOFT GROUND

Where embankments must be built over soft ground (such as soft clays, organic silts, or peat), stability and settlement of the fill should be carefully evaluated. In addition to the basic information listed under site considerations, the following information is to be provided in the project geotechnical report, where applicable.

### Embankment Stability

1. Stability of the embankment for minimum Factor of Safety (FOS) of 1.25 for sideslope stability and 1.30 for head slope stability of bridge approach embankments.

2. Shear strength of the foundation soil from lab testing and/or field vane shear.

3. Stability analysis calculations using c' values > 0 (from 5 to 15 kPa) or values considered appropriate for the different types of soils being analyzed are also required.

4. If the proposed embankment does not provide minimum factors of safety given above, recommendations for feasible treatment alternates which will increase the factor of safety to the acceptable minimum are required. Examples of recommendations are: change in alignment, lower grade, stabilizing counterberms, excavate and replace weak subsoil, stage construction, light-weight fill, geotextile fabric reinforcement, etc.

5. Cost comparisons of treatment alternates and recommendation of specific alternate.

### Settlement of Subsoil

1. Determination of consolidation properties of fine grained from laboratory consolidation tests.

2. Estimate of settlement amount and time rate of settlement.

3. Recommendations made to remove the settlement before the bridge abutment is constructed (waiting period, surcharge, or wick drains).

4. Proposed geotechnical instrumentation to monitor fill stability and settlement, with detailed recommendations provided on the number, type, and specific locations of the proposed instruments.

### Construction Considerations

1. Recommendation for excavation and replacement of unsuitable shallow surface deposits (muskeg, topsoil) with vertical and lateral limits of recommended excavation.

2. Plan and cross-section of surcharge treatment, if recommended.

3. Instructions or specifications concerning instrumentation, fill placement rates and estimated times for the contractors.

4. Recommendation for disposal of surcharge material after the settlement period is complete.

5. Details of wick drains, layout on plan and cross-section, if recommended.

# 4 STRUCTURE FOUNDATION CONSIDERATIONS

In addition to the basic information provided under Site Investigation Considerations, if pile support is recommended or given as an alternative, conclusions/recommendations must be provided in the geotechnical report for the following where applicable.

1. Recommended pile type (displacement, non displacement, pipe pile, concrete pile, H-pile, etc.) with valid reasons given for choice and/or exclusions.

2. Suitability and economy of recommended pile type(s).

3. Recommended factored and unfactored loads based on Ultimate Limit State Design considerations.

4. Estimated pile lengths and estimated tip elevations.

5. Settlement of piles and pile groups.

6. If a specified or minimum pile tip elevation is recommended, provide a clear reason for the required tip elevation, such as underlying soft layers, scour, downdrag, piles uneconomically long, etc.

7. Verification by analysis that the recommended pile section can be driven to the estimated or specified tip elevation without damage (especially applicable where dense gravel-cobble-boulder layers or other obstructions have to be penetrated).

8. Soil parameters to allow the structural engineer to evaluate lateral load capacity of piles where lateral load is an important design consideration.

9. For pile supported bridge abutments over soft ground.
   - Estimate of abutment pile downdrag load and evaluation of pile capacity based on design concepts outlined in the Canadian Foundation Engineering Manual.
   - Estimate of amount of abutment rotation that can occur due to lateral squeeze of soft ground.

10. Construction Considerations:
    - Pile driving details such as boulders or obstructions which may be encountered during driving
    - need for pre-augering, jetting, pile tip reinforcement, driving shoes, etc.
    - Excavation requirements - safe slope for open excavations, and need for sheeting or shoring. Fluctuation of groundwater table.
    - Evaluation of the effects of pile driving operations on adjacent structures, such as protection against damage caused by footing excavations or pile driving vibrations.
    - The need for pre-construction condition survey of adjacent structures to prevent unwarranted damage claims.
    - The need for pile driving control such as dynamic testing or wave equation analysis.

DRILLED SHAFTS

1. Recommendation of shaft diameter(s) and length(s) based on an analysis using soil parameters for side friction and end bearing.

2. Estimate of settlement for factored and unfactored loads.

3. Where lateral load capacity of shaft is an important design consideration, are p-y (load vs. deflection) curves or soils data provided which will allow the structural engineer to evaluate the lateral load capacity of shafts.

4. Construction Considerations:
   - Evaluation of construction methods - dry, slurry, and casing methods where applicable.
   - If casing will be required, can casing be pulled as shaft is concerted (this can result in significant cost savings on very large diameter shafts).
   - If artesian water was encountered in explorations, have design provisions been included to handle it (such as by requiring casing and trim seal).
   - Will boulders be encountered? Note: if boulders will be encountered, then the use of shafts should be seriously questioned due to construction installation difficulties and resultant higher costs the boulders can cause).

SPREAD FOOTINGS

1. If spread footings are not recommended for foundation support, are reasons for not using them discussed?

2. If spread footings supports are recommended the following are required:
   - Recommended bottom of footing elevation and reason for recommendation (e.g., based on frost depth, estimated scour depth or depth to competent bearing material).
   - Recommended factored and unfactored soil or rock bearing capacities.
   - Estimated footing settlement and time of settlement for factored and unfactored loads.
   - Recommendations for abutments to be placed on the bridge approach fills.
   - Gradation and compaction requirements for select approach fill and backwall drainage material.

3. Construction Considerations:
   - Adequate description of materials on which the footing is to be placed so the bridge inspector can verify that the material is acceptable.
   - Excavation requirements included for safe slopes in open excavations, need for sheeting or shoring, etc.
   - Fluctuation of the groundwater table.

RETAINING WALLS

1. Recommended soil strength parameters and groundwater elevation for use in computing wall design, lateral earth pressures and factor of safety for overturning, sliding, and external slope stability.

2. Acceptable reasons given for the choice and/or exclusion of certain wall types (gravity, reinforced soil, tieback, cantilever, etc).

3. Analysis of the wall stability with minimum acceptable factors of safety against overturning (FOS = 2.0), sliding (FOS = 1.5), and external slope stability (FOS = 1.5).

4. Estimated total settlement, differential settlement, and time rate of settlement if wall is to be placed on compressible foundation soils.

5. Determination of differential movement for compressible foundation soils.

6. Drainage details including materials and compaction to be provided.

7. Construction Considerations:
   - Excavation requirements - safe slopes for open excavations, need for sheeting or shoring.
   - Fluctuation of groundwater table.

# 5 LANDSLIDE REMEDIATION CONSIDERATIONS

Reporting and site investigation requirements for landslide investigation and assessment will follow the guidelines outlined in Sections 1and 2. In addition to those basic requirements the following information must be included in the geotechnical report on landslide analysis and/or remediation.

1. Site plan and scaled cross-section showing ground surface conditions both before and after failure.

2. Past history of the slide area, including movement history, summary of maintenance work and costs, and previous corrective measures taken.

3. Summary of results of the site investigation, field and lab testing, and stability analysis, including cause(s) of the slide.

4. Detailed slide features (including location of ground surface cracks, head scarp, and toe bulge) shown on the site plan.

5. Cross-sections used for stability analysis including the soil profile, water table, soil unit weights, soil shear strengths, and failure plane shown as it exists.

6. Slide failure plane location determined from slope indicators.

7. For an active slide, soil strength along the slide failure plane back-figured using a Factor of Safety (FOS) equals to 1.0 at the time of failure.

8. Proposed correction alternate (typical correction methods include but not limited to berms, shear key, rebuild slope, surface drainage, subsurface drainage - interceptor, drain trenches or horizontal drains and retaining structures).
   - Cross-section of proposed alternate
   - Estimated safety factor
   - Estimate cost
   - Advantages and disadvantages

9. Recommended correction alternate(s) which provide a minimum FOS of 1.25.

10. Use of horizontal drains as part of slide correction if the subsurface investigation located definite water bearing strata that can be tapped with horizontal drains.

11. Use of toe berm to stabilize an active slide must confirm that the toe of the existing slide does not extend beyond the toe of the proposed berm.

12. Construction Considerations:
    - Determination of "during construction backslope FOS" when proposed correction will require excavation into the toe of active slide area (such as for buttress or shear key).
    - Stage construction considerations when excavation FOS is near 1.
    - Consideration of fluctuation of groundwater table.
    - Recommendation for stability of excavation backslope to be monitored.
    - Description and specification of special construction features, techniques and materials.

# 6 GEOTECHNICAL CONSIDERATIONS FOR GRADING DESIGN

## GENERAL

Geotechnical investigations to assist in the selection of the most desirable gradeline for highway grading projects are normally conducted through shallow testhole drilling methods inside and immediately outside of the proposed roadway prism.

Testhole drilling may be supplemented by backhoe testpits where ground conditions dictate the necessity for a closer examination of the condition of the subsoils. Care must be exercised in testpitting operations since testpits within the roadway prism can result in weak zones if the pits are not properly compacted during backfill operations. A small size backhoe bucket should be used for roadway testpitting to minimize the extent of disturbance of the surrounding area.

Equipment used for testhole drilling and testpitting can be either on tracks or wheels. The most appropriate type of equipment for a particular project must be chosen with consideration for the terrain and time of year the investigation is to be undertaken.

## REQUIREMENTS

### 1. Testholes

Testholes are generally drilled at the intervals of approximately 200 m on alternate sides of the centre-line of the proposed alignment. These general guidelines must be reviewed in relation to an engineering assessment of the project. Sampling intervals can then be adjusted to suit the needs. The testholes shall be located to most effectively expose the character of the soils in cut areas and underlying embankments especially through soft/muskeg areas. Guidelines on the location of testholes for some frequently encountered situations are given in the AT&U Materials Manual (MEB 1), Section 5, "Preliminary Soils Investigation", copy included in Appendix D. Soil sampling proposals which differ significantly from the guidelines should be submitted to the Regional Construction Manager/Prime Consultant, along with the rationale for the changes.

Some general requirements for testhole drilling are as follows:

- Drill testholes to at least 2 metres below the proposed gradeline in cut sections.

- Drill testholes to at least 2 metres below the existing grade in fill sections. For soft ground, the depth of exploration should be dictated by stability and settlement considerations. Where subsoil conditions are fairly homogeneous and rapid construction of the fill is anticipated, the depth of exploration can be assessed from a consideration of the undrained shear strength of the subsoil and height of embankment to be constructed.

- For rapid construction, the safe height of embankment in metres may be estimated as 0.2 times the undrained shear strength of the subsoil, obtained for clay soils, using a pocket penetrometer. The pocket penetrometer value is divided by 2 for shear strength determination.

- For a fill height larger than that obtained for rapid construction, stage construction may be required. However, this requires an assessment of the stability and settlement considerations of the subsoils. The investigation and testing should be appropriate to assess this potential, if and when required.

- Investigate muskeg areas to determine the nature and thickness of the muskeg and underlying soils. The depth and character of the investigation shall allow for the settlement and stability considerations of the embankment, muskeg and underlying soils to properly evaluate sideslope design, berm requirements, geotextile requirements, removal of muskeg by excavation or displacement, floating fill construction techniques (stage loading and duration). Muskeg probing, testpitting, and vane shear can be used as primary or supplementary methods of investigation.

- Investigate potential borrow areas with a minimum of two (2) testholes each drilled to a depth of one (1) metre below the depth of proposed borrow excavation. Sufficient testholes or testpits are, however, required to provide both an area and depth evaluation of the borrow source. Investigation of borrow sources should be done at locations that will provide as far as practicable the optimum haul conditions in relation to the gradeline design. It may be prudent to discuss the investigation of borrow sources with the local public authorities prior to undertaking this exercise, especially where adjacent lands are all privately owned.

- Investigate rock areas to identify the type and quantities for determination of the most economical roadway design. Possible rock areas to be identified during the reconnaissance of the site, through airphoto interpretation and geological assessment, or from historical records of construction, where applicable. Prior to undertaking a detailed investigation of any rock areas, the scope of the investigation can be reviewed with the AI Regional Construction Manager for guidance to ensure that Department's standards are maintained.

- Existing roadways must be investigated for the quality of the existing embankment. Where roadways are to be widened, the nature of the subsoils in the areas to be widened need to be properly assessed. The relationship of the grade of the existing roadway to the proposed gradeline would dictate the extent of investigation other than the usual requirements especially through soft areas like creek/lake crossings.

- Sidehill cut and fill situations must be thoroughly investigated to determine groundwater seepage conditions and the depth of unsuitable material on existing slopes. Unsuitable materials must be well defined since their removal is essential to the stability of the superimposed embankment fill. Seepage conditions are very prevalent where sidehills form slopes of river and creek valleys. Since moisture movement is seasonal, the installation of piezometers (standpipe or pneumatic types) coupled with visual observations, and testpitting of the sidehill where feasible would allow groundwater conditions to be properly assessed. Such assessment is necessary for proper design of the superimposed embankment fills.

- Testholes should be drilled through the sidehill to about two (2) metres below the toe of the proposed embankment to ensure that the soil stratigraphy and seepage conditions are properly evaluated. Alternatively, and where practical, testpits are much preferred since subsurface seepage conditions can generally be observed in a short time.

2. Additional Investigation

Additional investigation, including deep drilling, undisturbed sampling and rock coring may be required depending on the topographical features of the alignment, proposed cross-sectional elements, and the subsoil conditions and characteristics encountered. The need for deep drilling and undisturbed sampling, and any special investigations should preferably be identified during the site reconnaissance. Costs for any additional investigation must be identified at the time of proposal submission.

Where existing roadways are to be widened testhole drilling should be done within the existing roadway at least 3 locations per kilometre to assess the nature of the existing roadway. This will assist in the design of the widened portion of roadway. Existing mosaics can provide historic borehole information and should be reviewed.

Where existing roadways are to be realigned drilling of the existing embankment at a few locations should be undertaken since this information could assist in design and construction of the relocated roadway.

In general, the characteristics of the existing roadway should be utilized as much as possible to aid in the design of the new or improved facility especially if the existing roadway is within similar terrain as the one in which the new or upward roadway is to be constructed. This concept is of particular importance in muskeg/soft ground areas and in hilly terrain where seepage considerations have to been looked at carefully with respect to roadway widening.

For borrow pit samples, sufficient Atterberg Limits shall be done in all cohesive borrow on a sufficient number of samples to allow evaluation of the borrow material for use as a road building fill. Field moisture contents and estimated standard Proctor optimum moisture content and maximum dry density are essential characteristics to aid in evaluating borrow suitability and must be provided in test results.

For non-cohesive borrow material (sands, gravels, silts, silty sands and other combinations of materials on which standard Atterberg Limits testing cannot be undertaken) the Alberta Transportation and Utilities Family of Curves is to be used as a guideline for determining the optimum moisture and estimated standard Proctor maximum dry density. Both the field moisture and visual soil classification can be used in the assessment of the best optimum moisture and maximum density characteristics.

## 3. Geotechnical Investigation and Assessment for Bridge and Culvert Sites

In many instances upgrading of existing roadways or now alignment construction may require geotechnical investigation and assessment of bridge and culvert foundations, and approach fill stability. The technical requirements for undertaking and reporting on these investigations are provided in Sections 1 and 2.

It is important that the investigation of the sub-structure foundations and approach fills is not done in isolation of the remaining roadway alignment since materials to construct the approach fills, and ground conditions just beyond the approach fills may have an overall influence on structure performance.

If such detailed investigation was not done during the geotechnical investigation for the gradeline design, then this must be undertaken at the time of the structure investigation so that a complete evaluation of the site can be undertaken.

## 4. Laboratory Testing

While samples are required to be taken at intervals of 200 m along the alignment generally, testing of all samples would be time consuming and expensive. However, a sufficient number of samples as required to be tested to provide sufficient information on the ground conditions to assist the geometric and geotechnical designer to access the disposition of the design gradeline and the behaviour and performance of the grade. In addition, the soils information on mosaics is essential for bidding, construction, and maintenance purposes. Sufficient testing information is required to allow the contractor to understand the soils available for construction and the ground conditions throughout the alignment within the work area so that a proper bid would be provided and the project executed satisfactorily.

Laboratory test results shall include:

- Visual description
- Field moisture content
- Atterberg Limits according to current ASTM procedures
- Washed sieve analysis, including the 5000, 1250, 400, 160, and 80 metric sieves
- Classification according to the Unified Soil Classification System (USCS) as modified by the Prairie Farm Rehabilitation Association (PFRA), based on laboratory test results

The following information shall also be included as part of the summary of test results:

- Estimates of standard Proctor maximum dry density and optimum moisture content based on Alberta Transportation and Utilities Charts.

- Plasticity Index and Liquidity Index.

As mentioned previously, to minimize the number of tests the following approach is recommended:

- Visual description and classification, and field moisture content tests shall be conducted on all soil samples. The frequency of Atterberg Limits and washed sieve analysis testing may be reduced when the samples are visually identified as being similar and noted as such on mosaics. Complete Atterberg Limits, sieve analysis and field moisture content testing shall be done at a minimum of three (3) testhole or testpit locations within a kilometre length of grade.

- For estimation of testing requirements, samples for sieve analysis, Atterberg Limits and field moisture contents shall be provided for three (3) testhole locations over a one (1) km length of alignment as follows:

  For an estimated 2 m depth of hole samples would be normally taken at 1 m intervals of depth. While moisture content on all samples are required, Atterberg Limits and sieve analysis are required only on selected samples. For estimation purposes, one (1) sieve analysis and one (1) Atterberg Limit test for a 2 m hole should be used. The number of sieve analyses and Atterberg Limits can be further reduced when the samples are scrutinized following the field drilling. If a particular situation warrants increased testing in excess of estimates this request will have to be approved.

5. Additional Testing

Additional testing may be required depending on local conditions encountered. The type of tests to be done as additional work should be clearly spelled out at the time of proposal submission.

6. Presentation of Soils and Rock Information on Mosaics

Soils descriptions on the mosaics are to consist of the principal soil types without the consistency descriptions such as hard, soft, stiff, etc. The terminology 'till' which embodies glacial materials consisting of silt, clay, sand should not be placed on mosaics, instead the description according to the Unified Soil Classification System modified by the PFRA should be used. However, these descriptions, as well as the consistency of the soils, should be shown on the field logs obtained during the investigation. These field descriptions are of importance to the interpretation of soil conditions at the time of the investigation.

Where the rock or rock type materials such as shale, sandstone, siltstone are encountered, only the field visual descriptions (shale, sandstone) must be shown on the mosaics logs with the corresponding graphic symbol. Note, however, that the soil classification determined through laboratory modification of the material would be reported as CH, CI, SM based on the Atterberg Limits testing. The rock report, however, would contain information on material quality and type. A note is to be made on the mosaics referring the contractor to the "Rock Report" for detailed information.

7. Reports

A Geotechnical report shall be prepared for the proposed route and shall summarize soils stratigraphy, test results and construction recommendations. The Geotechnical report shall form part of the documentation to be used by the Regional Construction Manager during construction.

The report shall address, where applicable, the following special conditions but shall not only be limited to these, if conditions dictate. Typical drawings of any recommended measures and any applicable recommendations are to be incorporated in the report.

- Pertinent topographic features (present condition and brief history of nearby roads or structures where similar conditions prevail).
- Seepage and groundwater conditions and the necessity for, and details of, any surface and subsurface drainage measures.
- Sidehill construction and associated stability considerations due to seepage and weak subsoil materials.
- Design of any pre-drainage measures for borrow sources, large cutslopes and slough areas.
- High embankment and high cutslope stability and associated considerations.
- Muskeg and soft ground design and construction considerations including pre-loading and stage construction.
- Design considerations of new embankment fill and/or grade widening embankment fill in slough areas, need for berms, consolidation and stability considerations.

- Condition and disposition of existing roadway embankments.
- Culvert foundation treatment and type of culvert for the prevailing soil conditions and any special procedures to be implemented at the time of excavation for culvert bed preparation.
- Special soil problems such as bentonite, frost susceptibility, potentially expansive materials, influence of coal seams and topsoil problems in existing and new alignments.
- Anticipated influence due to likely change of soil moisture conditions, especially in silty soils, between the time of investigation and the time of grade construction.
- Evaluation of borrow pit materials and associated recommendations.
- Swell and shrinkage factors for soils, rock materials, and soil/rock combinations.
- Water for compaction requirements, and compaction considerations of silt and sandy soils.
- Stabilization of silt and sandy soils where these soils form the roadway running surface.
- Use of lime as a drying aid and stabilizer, the quantity of lime required and recommendations on its effective use.
- Combined settlement and shrinkage factor for embankments on muskeg. Past experiences are available from the Technical Standards Branch, AT&U.
- Minor relocations and adjustment of grades to avoid unstable conditions or rock or boulder protuberances.
- Rock investigation particulars such as type of rock, extent of weathering, height of cut or slope, attitude and extent of fracture, description of existing rock cuts in nearby similar material, and rock fall problems in areas of similar materials. A rock investigation report is required separate from the geotechnical report for the use by contractors during bidding. Samples of this report can be viewed at the Regional Construction Manager’s office. The Rock Report should also contain recommendations on how the rock excavation is to be paid for, any special procedures to be followed and precautions needed.
- Landslides and their impact on route stability, recommendations or stabilization measures where necessary.
- Presence of utility lines, especially high pressure gas lines and the impact of embankment and cutslope constructions on the performance of these lines. Lateral and vertical deformations are to be assessed where the roadway has to unavoidably encroach on these lines.
- Erosion potential and recommendations to prevent or minimize erosion. Design of any special temporary or permanent measures is to be included. Refer to the department’s *Erosion Control Reference Material,* 1998.

- Methods for the best use of soil materials including the direction of haul where pertinent.

- Geotextile and geomembrane uses, quantities, types, costs and specifications. The use of geotextiles and geomembranes (geosynthetics, in general) requires substantiating why these materials are necessary from an engineering assessment point of view. This assessment is to incorporate, in addition to experience and engineering judgement, calculations which would demonstrate that such materials would be very essential to the immediate and long term performance of the roadway. In undertaking this assessment, factors such as the type of facility, its future use, risk, and maintenance aspects are to be taken into consideration. Geosynthetics can be costly and all efforts must be made to demonstrate that these materials are warranted.

- Instrumentation and monitoring requirements.

- Detailed Special Provisions for geotechnically related measures. Copies of Special Provisions currently in use within the Department can be requested from Technical Standards Branch, AI or downloaded from the department website, www.infras.gov.ab.ca.

8. Gradeline Review Process

- During the course of the gradeline design by a prime consultant, the geotechnical sub-consultant shall review the proposed gradeline for compatibility with the prevailing and/or anticipated subsoil conditions.

- For roadways in virgin terrain or existing unpaved roadways, discussions with nearby residents and local roadway officials may provide information on ground conditions that are not readily apparent from subsurface drilling.

- For any planned review meetings with AI staff, a copy of the geotechnical report and a complete set of mosaics with all soils information included along with relevant cross-sections must be submitted at least one week prior to the meeting date.

- It is the responsibility of the geotechnical sub-consultant and/or prime consultant to present the geotechnical issues at the review meeting and respond to any questions that may be raised by others concerning any geotechnical issues.

9. Project Summary Reports

Projects summary reports (following completion of grading construction), must address, where applicable, all geotechnical concerns related to the project that were included in the geotechnical recommendations or that may have occurred during construction and not anticipated at the design stage.

Some typical examples of concerns are noted below, but are not only limited to these.

- Muskeg/Soft Soil Conditions

  If preloading was done, what were the station limits? If geotextile was used, what were the station limits, type of geotextile and quantity used? If any muskeg was excavated, where was it placed? Any other construction precautions followed, such as muskeg ditching, stage construction, and their success?

- Springs, Seepage Zones or High Water Table in Cut or Fill Areas

  Any drainage trenching done, limits of trenching and success of operation. Any icing locations in the ditches, which may require future attention.

- Creek or Lake Crossings, Culvert Locations

  Soft bed conditions at culvert crossings and treatment used. Any occurrence of slides while excavating for the bed preparation, and what remedial measures were taken. Any special measures implemented for grade construction across lakes or adjacent to lake shores. Locations of any diverted creeks, which may have a bearing on future performance of the fill in those locations.

- Deep Cut Areas

  Any benching followed. Occurrence of any springs and measures adopted. Any rock encountered.

- Rock Cut Areas

  Estimated versus actual quantities. Type of rock. How variable the rock layers were with depth. What type of procedures the contractor followed to excavate the rock. How the payment aspect was dealt with. Any problems with the contractor, and how they were resolved.

- High Fill Areas, Steep Side Hill Construction

  Any special drainage measures required in order not to block the natural drainage pattern in the fill areas.

- Gravel or Coal Seams

  Any necessity of removing the coal, and the quantity so removed. Any seepage from the coal seams and how it may affect the grade performance. Any remedial measures implemented.

- Borrow Materials

  How wet were these materials? Were any special measures necessary to aid drying? What were the shrinkage values used in the design and those actually obtained? How were borrow pits reclaimed?

- High Pressure Pipe Line Crossings Across the Roadway or Along the Backslopes or Sideslopes

  What precautions were taken at the design stage to minimize movements in the pipeline system due to the gradeline construction in their vicinity, and their success?

- Disposal of Waste Material

  What materials were categorized as waste and why? How were these materials utilized? Were they disposed outside of the right-of-way or utilized within the roadway prism?

- Other Geotechnical Concerns

  Any problems that may be geotechnically related and any opinions that may be essential to improvement in grading designs are most welcome.

- Future Pavement Design and Construction

  Problems or areas of concern that have to be taken in consideration in future pavement design and construction, such as weak subgrade areas, high groundwater levels and frost susceptible materials.

## REFERENCE

1. Alberta Infrastructure (2000). *Standard Specifications for Highway Construction*

2. Alberta Infrastructure (1998). *Engineering Consultant Guidelines for Primary Highway Projects*

3. Alberta Infrastructure (1998). *Engineering Consultant Guidelines for Secondary Highway Projects*

4. Alberta Infrastructure (1999). *Traffic Accommodation in Work Zones* (Revision available April 2001)

5. AT&U (1990*). Materials Manual (MEB 1)*

Note: All current documents may be purchased from Alberta Transportation or viewed and downloaded from the website, www.infras.gov.ab.ca.

# A REVISED SUBSISTENCE AND TRAVEL RATES AND ALLOWANCES

(Effective April 1, 2001)

# NOTICE TO GOVERNMENT OF ALBERTA EMPLOYEES

## REVISED SUBSISTENCE AND TRAVEL RATES AND ALLOWANCES

Effective April 1, 2001

This notice shows the revised, **most frequently claimed travel rates** and allowances which are effective April 1, 2001.

### Business Kilometre (Km) Rates

| Class | In Fiscal Year (April 1 to March 3 | |
|---|---|---|
| | First 15,000 km | Each Km Thereafter |
| **Class A.** Government business travel primarily in Central and southern Alberta south of specified boundaries. | 33.5 cents per km. | 27.5 cents per km. |
| **Class B** Travel primarily in Northern Alberta. | 34.5 cents per km. | 28.5 cents per km. |

### Business Insurance for Private Vehicle

- Maximum of $260.00 per year if additional vehicle insurance premium is paid for Government business travel.

### Meal Allowances (no receipts required)

- Breakfast $6.60
- Lunch $8.40
- Dinner $15.25

Note: meal allowances include gratuity and GST; therefore these may not be claimed when claiming a meal allowance.

### Overtime Meals

- Purchase of a meal during or immediately following completion of an authorized overtime period greater than 2 hours beyond the normal daily hours – up to $7.60.

## Personal Expense Allowance (for each full 24-hour period on travel status)

- In Canada $ 5.25
- Outside Canada $10.50

## Accommodation Expenses

- Private accommodation; no receipt required - $14.70 per night; or
- Actual accommodation costs; receipt required – please request Government rate.

## Fort McMurray Allowance

- A temporary allowance of $400 per month for each Government employee residing and working in Fort McMurray. This allowance will be reviewed in two years.

The Subsistence Travel and Moving Expenses Regulation, Directives and Guidelines for employees to claim travel expenses are on the PAO website at www.gov.ab.ca/pao/ . These will be updated effective April 1, 2001 to incorporate all changes to the travel rates and allowances.

January 2001

# B SAMPLE FORMS

ALBERTA
TRANSPORTATION
& UTILITIES

# FIELD TEST HOLE LOG

DATE JAN 19-20/93

PROJECT: B.F. 79440 ( Hwy 16 & 41 ) SEC. ___ T.P. ___ RG. ___ MER ___

STATION 10.25 OFFSET -5 m HOLE NO. 1

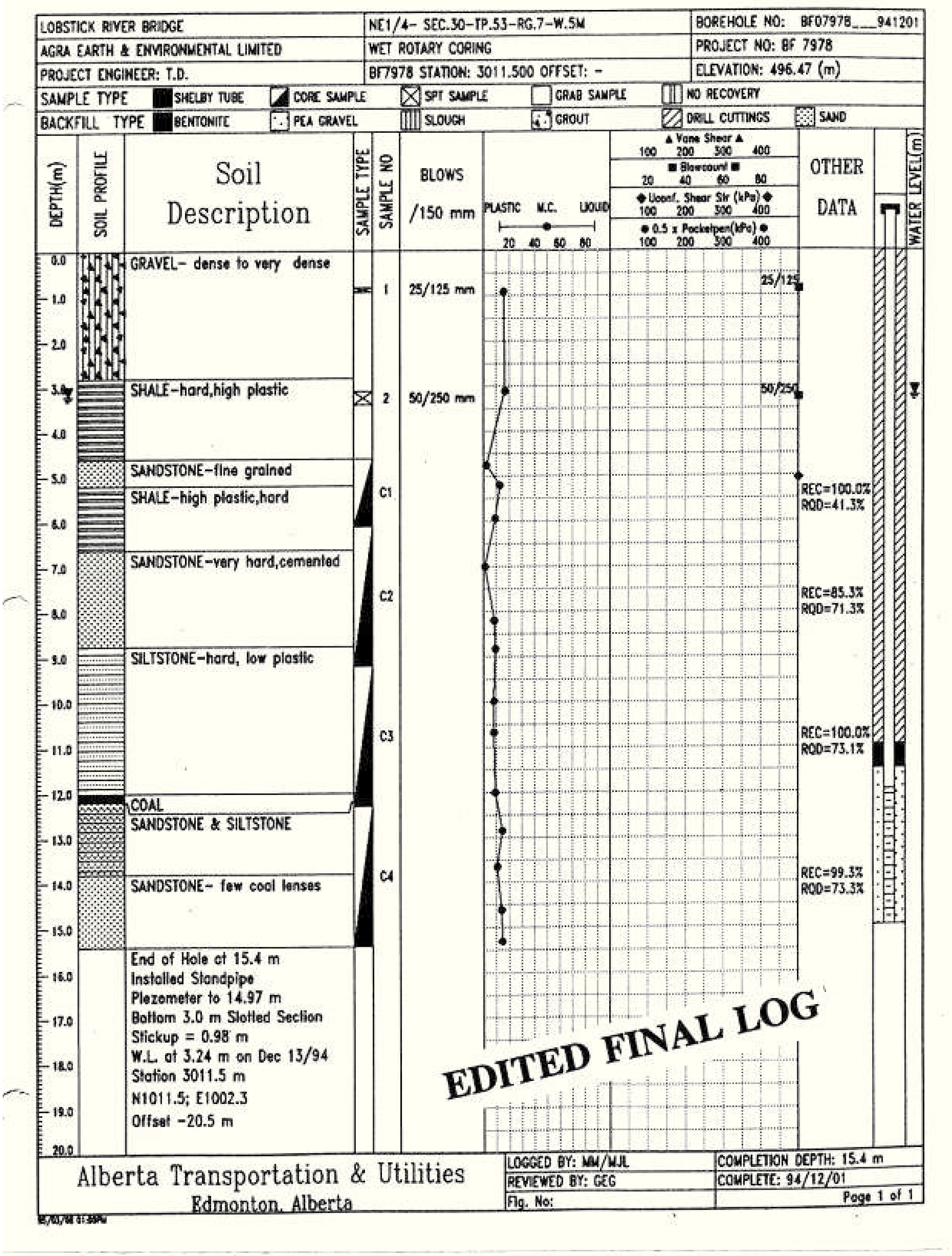
SURFACE ELEVATION 968.450 m DATUM ___

ENGINEER M. PARITI TECHNICIAN B. WIENS

CONTRACTOR MOBILE AUGERS & RESEARCH EQUIPMENT B66 HOLLOW STEM

START TIME 8:15 Am COMPLETION TIME 4:00 pm

| DEPTHS (m) FROM | AT | TO | SOIL DESCRIPTION AND REMARKS TYPE, COMPOSITION, CONSISTANCY, RELATIVE DENSITY, STRUCTURE PLASTICITY, COLOUR, INCLUSIONS, WATER, ETC. | SAMPLING SAMPLE NUMBER | TYPE | FROM-TO (m) | "N" OR % RECOVERY |
|---|---|---|---|---|---|---|---|
| 0 | | 1.5 | SILT, SANDY (FINE), LT. BROWN, | | | | |
| | | | DRY-FROZEN TO APPROX. 2.5m FROZEN | 12 | SPT | 0.76 | 10-27-24 |
| | | | | | | - 1.22 | |
| 1.5 | | | CLAY (TILL) SANDY + SILTY, FIRM | | | | |
| | | | STIFF, LOW-MED PLASTIC, | 13 | SPT | 2.29 | 7-11-14 |
| | | | BROWN, PEBBLES, NEAR OPT. PRTIALLY | PP= 4.0 | | -2.74 | |
| | | | MOIST PP=4.5 FROZEN | 33389 | SHELBY | 3.81 | 100% REC. |
| | 4.9 | | -BECOMES VERY SILTY, LOW | | | -4.42 | |
| | | | PLASTIC, STIFF BROWN | 14 | SPT | 5.33 | 7-11-20 |
| | 9.4 | | -BECOMES DR. GREY | | | -5.79 | |
| | | 12.4 | | NR | SHELBY | 6.86 | 10% REC |
| | | | | | | -7.47 | |
| 12.4 | | | CLAY (TILL), SANDY + SILTY, FIRM, PP=4.5 | 15 | SPT | 8.38 | 8-14-20 |
| | | | MED PLASTIC, GRAY, ABOVE OPT. | | | -8.84 | |
| | | 19.8 | MOISTURE PP=4.0 | 35289 | SHELBY | 9.91 | 100% |
| | | | | | | - 10.51 | |
| 19.8 | | | SAND (FINE), SILTY, | 16 | SPT | 11.43 | 7-12-19 |
| | | | DENSE, DAMP, LT. BROWN | | | -11.89 | |
| | | | PP=1.75 | 17 | SPT | 12.95 | 6-9-12 |
| | 21.5 | | -BECOMES WET, GREY | | | - 13.41 | |
| | | | PP = 2.5 | 34589 | SHELBY | 14.48 | 100% REC. |
| | | | | | | - 15.09 | |
| | | | PP = 2.25 | 18 | SPT | 16.00 | 7-12-16 |
| | | | | | | - 16.46 | |
| | | | PP = 4.0 | 19 | SPT | 17.52 | 11-15-18 |
| | | | | | | -17.98 | |
| | | | | NR | SPT | 19.05 | 50/25mm |
| | | 29.9 | TERMINATED BOREHOLE @ 29.9m | | | - | |
| | | | | 20 | SPT | 20.57 | 33-63/150mm |
| | | | | | | -20.88 | |
| | | | | 21 | SPT | 22.10 | 14-40-50 |
| | | | | | | -22.56 | |
| | | | | 22. | SPT | 23.62 | 10-18-40 |
| | | | | | | - 24.08 | |
| | | | | 23 | SPT | 26.67 | 72/150mm |
| | | | | | | -26.82 | |
| | | | | 24 | SPT | 29.72 | 18-61/150mm |
| | | | | | | - 29.87 | |
| | | | BAG # 91-2050 and 91-2065 | | | | |


LOBSTICK RIVER BRIDGE
AGRA EARTH & ENVIRONMENTAL LIMITED
PROJECT ENGINEER: T.D.
NE1/4- SEC.30-TP.53-RG.7-W.5M
WET ROTARY CORING
BF7978 STATION: 3011.500 OFFSET: -
BOREHOLE NO: BF0797B___941201
PROJECT NO: BF 7978
ELEVATION: 496.47 (m)
SAMPLE TYPE
SHELBY TUBE
CORE SAMPLE
SPT SAMPLE
GRAB SAMPLE
NO RECOVERY
BACKFILL TYPE
BENTONITE
PEA GRAVEL
SLOUGH
GROUT
DRILL CUTTINGS
SAND
DEPTH(m)
SOIL PROFILE
Soil Description
SAMPLE TYPE
SAMPLE NO
BLOWS /150 mm
PLASTIC
M.C.
LIQUID
20 40 60 80
▲ Vane Shear ▲
100 200 300 400
■ Blowcount ■
20 40 60 80
◆ Uconf. Shear Str (kPa) ◆
100 200 300 400
● 0.5 x Pocketpen(kPa) ●
100 200 300 400
OTHER DATA
WATER LEVEL(m)
GRAVEL- dense to very dense
1
25/125 mm
25/125
SHALE-hard,high plastic
2
50/250 mm
50/250
SANDSTONE-fine grained
SHALE-high plastic,hard
C1
REC=100.0%
RQD=41.3%
SANDSTONE-very hard,cemented
C2
REC=85.3%
RQD=71.3%
SILTSTONE-hard, low plastic
C3
REC=100.0%
RQD=73.1%
COAL
SANDSTONE & SILTSTONE
SANDSTONE- few coal lenses
C4
REC=99.3%
RQD=73.3%
End of Hole at 15.4 m
Installed Standpipe
Piezometer to 14.97 m
Bottom 3.0 m Slotted Section
Stickup = 0.98 m
W.L. at 3.24 m on Dec 13/94
Station 3011.5 m
N1011.5; E1002.3
Offset -20.5 m
EDITED FINAL LOG
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
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17.0
18.0
19.0
20.0
Alberta Transportation & Utilities
Edmonton, Alberta
LOGGED BY: MM/MJL
REVIEWED BY: GEG
Fig. No:
COMPLETION DEPTH: 15.4 m
COMPLETE: 94/12/01
Page 1 of 1

**ALBERTA**
**TRANSPORTATION**
**& UTILITIES**

# FIELD TEST HOLE LOG

DATE ________

PROJECT: ______________________ SEC.____ T.P. ____ RG. ____ MER____

STATION ______________ OFFSET ________ HOLE NO.________

SURFACE ELEVATION ______________________ DATUM ____________

ENGINEER ______________________ TECHNICIAN ____________

CONTRACTOR______________________ EQUIPMENT ____________

START TIME ______________ COMPLETION TIME ____________

| DEPTHS (m) | | | SOIL DESCRIPTION AND REMARKS | SAMPLING | | | |
|---|---|---|---|---|---|---|---|
| FROM | AT | TO | TYPE, COMPOSITION, CONSISTENCY, RELATIVE DENSITY, STRUCTURE PLASTICITY, COLOUR, INCLUSIONS, WATER, ETC. | SAMPLE NUMBER | TYPE | FROM–TO (m) | "N" OR % RECOVERY |
| | | | | | | | |

WATER LEVEL: ______________

# ALBERTA
TRANSPORTATION & UTILITIES

## SOIL SURVEY SAMPLE IDENTIFICATION

### JOB IDENTIFICATION

| PROJECT NO. | FROM: | TO: | DISTRICT NO. | DISTRICT OFFICE | PROJECT MANAGER | PHONE NO. | TECHNOLOGISTS |
|---|---|---|---|---|---|---|---|
| SH686:01 | Slide – East Hill | | 14 | Twin Atria | Karl Li | 427–3101 | B. Wiens |

| SAMPLE NUMBERS FIRST | SAMPLE NUMBERS LAST | DATE SAMPLED | SAMPLE SOURCE | BAG NO. | JOB NO. | DESIGN DATE | REMARKS |
|---|---|---|---|---|---|---|---|
| | | 92–11–25 | SPT | | T310D | | |

| LAB SAMPLE NUMBER | TAG NO. | HOLE NO. | STATION | LOCATION | TOPSOIL DEPTH | SAMPLE DEPTH | TRANSITION DEPTH | FIELD REMARKS | FIELD VISUAL DESCRIPTION | INSTRUCTION |
|---|---|---|---|---|---|---|---|---|---|---|
| | 201 | 16 | 13 + 243.800 | 32.8m Lt | | 1.52 | | | | |
| | 202 | 16 | " + " | " | | 4.57 | | | | SAMPLE SOURCE: |
| | 203 | 16 | " + " | " | | 7.62 | | | | 1. SOIL SURVEY |
| | 204 | 16 | " + " | " | | 10.67 | | | | 2. SHELBIES |
| | 205 | 16 | " + " | " | | 13.72 | | | | 3. PENO |
| | 206 | 16 | " + " | " | | 16.76 | | | | 4. 80mm CORE |
| | 207 | 16 | " + " | " | | 18.29 | | | | 5. S.P.T. |
| | 208 | 16 | " + " | " | | 13.10 | | | | 6. OTHER |
| | 209 | 13 | 13 + 294.800 | 33.3m Lt | | 3.05 | | | | |
| | 210 | 13 | " + " | " | | 6.40 | | | | ALL DEPTHS: |
| | 211 | 13 | " + " | " | | 9.14 | | | | TOPSOIL |
| | 212 | 13 | " + " | " | | 14.93 | | | | SAMPLE |
| | 213 | 13 | " + " | " | | 17.07 | | | | TRANSITION |
| | 214 | 13 | " + " | " | | 18.29 | | | | RECORD TO TWO |
| | 215 | 13 | " + " | " | | 21.34 | | | | DECIMAL PLACES |
| | 216 | 13 | " + " | " | | 24.38 | | | | |
| | 217 | 13 | " + " | " | | 27.43 | | | | FOR ADDITIONAL |
| | 218 | 13 | " + " | " | | 29.87 | | | | INSTRUCTIONS |
| | | | + | | | | | | | SEE SECT. 3.1.03 |
| | | | + | | | | | | | MEB & MATERIALS |
| | | | + | | | | | | | MANUAL |
| | | | + | | | | | | | |
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**ALBERTA**
**TRANSPORTATION & UTILITIES**

# SOIL SURVEY SAMPLE IDENTIFICATION

## JOB IDENTIFICATION

| PROJECT NO. | FROM: | TO: | DISTRICT NO. | DISTRICT OFFICE | PROJECT MANAGER | PHONE NO. | TECHNOLOGISTS |
|---|---|---|---|---|---|---|---|
| | | | | | | | |

| SAMPLE NUMBERS FIRST | LAST | DATE SAMPLED | SAMPLE SOURCE | BAG NO. | JOB NO. | DESIGN DATE | REMARKS |
|---|---|---|---|---|---|---|---|
| | | | | | | | |

| LAB SAMPLE NUMBER | TAG NO. | HOLE NO. | STATION | LOCATION | TOPSOIL DEPTH | SAMPLE DEPTH | TRANSITION DEPTH | FIELD REMARKS | FIELD VISUAL DISCRIPTION | INSTRUCTION |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | + | | | | | | | SAMPLE SOURCE:<br>1. SOIL SURVEY<br>2. SHELBIES<br>3. PENO<br>4. 80mm CORE<br>5. S.P.T.<br>6. OTHER<br><br>ALL DEPTHS:<br>TOPSOIL<br>SAMPLE<br>TRANSITION<br>RECORD TO TWO<br>DECIMAL PLACES<br><br>FOR ADDITIONAL<br>INSTRUCTIONS<br>SEE SECT. 3.1.03<br>MES 1 MATERIALS<br>MANUAL |
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| **ALBERTA**<br>TRANSPORTATION<br>& UTILITIES | **LABORATORY<br>TEST REQUEST** | FILE NO. SH 686:01 | JOB NO. T 310 D |
|---|---|---|---|
| | | SITE East Hill Slide | |
| | | DATE: | |
| | | REQUESTED | REQUIRED |
| | | PRIORITY | ENGINEER |

| LAB SAMPLE NUMBER | HOLE NUMBER | TOP OF HOLE ELEV. | STATION | LOCATION | DEPTH (m) | SAMPLE TYPE | ATTERBERG LIMIT | SIEVE ANALYSIS | MOISTURE CONTENT | UNCONFINED COMPRESSION | VANE SHEAR | POCKET PENETROMETER | CONSOLIDATION | TRIAXIAL COMPRESSION TEST | CORES | DIRECT SHEAR PEAK STR. PEAK & RESID. QUICK TEST | INSTRUCTIONS: |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 13 | 442.63 | 13 + 294.800 | 33.3m Lt | 1.22 | S | | | | | | | | | | | SAMPLE TYPE: |
| | 13 | " | " + " | " | 3.05 | P | | | | | | | | | | | S – SHELBY |
| | 13 | " | " + " | " | 4.27 | S | | | | | | | | | | | C – CORE |
| | 13 | " | " + " | " | 6.40 | P | | | | | | | | | | | B – BAG |
| | 13 | " | " + " | " | 7.92 | S | | | | | | | | | | | P – S.P.T |
| | 13 | " | " + " | " | 9.14 | P | | | | | | | | | | | |
| | 13 | " | " + " | " | 10.67 | S | | | | | | | | | | | TRIAXIAL TESTS: |
| | 13 | " | " + " | " | 13.10 | S | | | | | | | | | | | UU – UNCONSOLIDATED |
| | 13 | " | " + " | " | 14.93 | P | | | | | | | | | | | UNDRAINED |
| | 13 | " | " + " | " | 17.07 | P | | | | | | | | | | | CU – CONSOLIDATED |
| | 13 | " | " + " | " | 18.29 | P | | | | | | | | | | | UNDRAINED |
| | 13 | " | " + " | " | 21.34 | P | | | | | | | | | | | CD – CONSOLIDATED |
| | 13 | " | " + " | " | 24.38 | P | | | | | | | | | | | DRAINED |
| | 13 | " | " + " | " | 27.43 | P | | | | | | | | | | | P – WITH PORE |
| | 13 | " | " + " | " | 29.87 | P | | | | | | | | | | | PRESSURE |
| | 16 | 441.01 | 13 + 243.800 | 32.8m Lt | 1.52 | P | | | | | | | | | | | CORES: |
| | 16 | " | " + " | " | 3.05 | S | | | | | | | | | | | P – POINT LOAD INDEX |
| | 16 | " | " + " | " | 4.57 | P | | | | | | | | | | | C – COMPRESSIVE STR. |
| | 16 | " | " + " | " | 7.62 | P | | | | | | | | | | | A – ACID REACTION |
| | 16 | " | " + " | " | 9.14 | S | | | | | | | | | | | S – SLAKING TEST |
| | 16 | " | " + " | " | 10.67 | P | | | | | | | | | | | |
| | 16 | " | " + " | " | 12.19 | S | | | | | | | | | | | |

REMARKS

**ALBERTA**
TRANSPORTATION
& UTILITIES

# LABORATORY TEST REQUEST

| FILE NO. | JOB NO. |
|---|---|
| SITE | |
| DATE: | |
| REQUESTED | REQUIRED |
| PRIORITY | ENGINEER |

| LAB SAMPLE NUMBER | HOLE NUMBER | TOP OF HOLE ELEV. | STATION | LOCATION | DEPTH (m) | SAMPLE TYPE | ATTERBERG LIMIT | SIEVE ANALYSIS | MOISTURE CONTENT | UNCONFINED COMPRESSION | VANE SHEAR | POCKET PENETROMETER | CONSOLIDATION | TRIAXIAL COMPRESSION TEST | CORES | DIRECT SHEAR PEAK STR. PEAK & RESID. QUICK TEST |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
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INSTRUCTIONS:

SAMPLE TYPE:
- S – SHELBY
- C – CORE
- B – BAG
- P – S.P.T

TRIAXIAL TESTS:
- UU – UNCONSOLIDATED UNDRAINED
- CU – CONSOLIDATED UNDRAINED
- CD – CONSOLIDATED DRAINED
- P – WITH PORE PRESSURE

CORES:
- P – POINT LOAD INDEX
- C – COMPRESSIVE STR.
- A – ACID REACTION
- S – SLAKING TEST

REMARKS

**ALBERTA**
**TRANSPORTATION**
**& UTILITIES**
**MATERIALS ENGINEERING BR.**
**Geotechnical Services Section**

## SLOPE INDICATOR INSTALLATION RECORD

**PROJECT:** SH 686:01

**LOCATION:** Daishowa Bridge – East Headslope

**STATION:** 11+001.6

**DATE INSTALLED:** December 11, 1992

**TEST HOLE NO.** No. 20

**S.I. CASING NO.** No. 20

**OFF SET:** 8.0m Rt CL

**ELEVATIONS:**

**GROUND SURFACE** 319.74 metres

**TOP OF CASING** 320.71 metres

**TOTAL LENGTH S.I. CASING INSTALLED:**

30.5 metres

100 feet

**DRILL CONTRACTOR:** Hertz Drilling Ltd.

**DRILLER:** D. Hertz

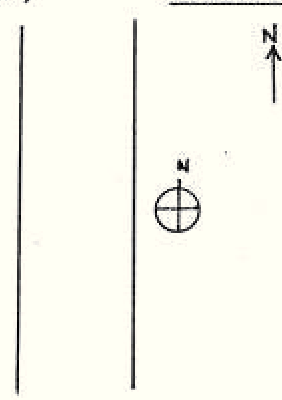
**CONSULTANT:** A. T. & U.

**TECHNOLOGIST:** B. Wiens

**GROUT MIXTURE & QUANTITY:** 3 – bags cement, 1 bag drill mud, to approx. 80 liters of water

**GROOVE ORIENTATION AT TOP OF CASING:**
**(Draw diagram indicating groove orientation reference point for slope indicator sensor.)**

N – S grooves set perpendicular to the centerline of the roadway




**COMMENTS: (Installation Procedures Etc.)** Drilled borehole to the desired depth – grouted borehole with the above mentioned grout mixtyre – set inclinometer casing into the grouted borehole.

ALBERTA
TRANSPORTATION
& UTILITIES
MATERIALS ENGINEERING BR.
Geotechnical Services Section

## SLOPE INDICATOR INSTALLATION RECORD

PROJECT: ______________________ DATE INSTALLED: ______________

LOCATION: ______________________ TEST HOLE NO. ______________

______________________ S.I. CASING NO. ______________

STATION: ______________________ OFF SET: ______________

ELEVATIONS:

GROUND SURFACE ______ metres

TOP OF CASING ______ metres

TOTAL LENGTH S.I. CASING INSTALLED:

______ metres

______ feet

DRILL CONTRACTOR: ______________ CONSULTANT: ______________

DRILLER: ______________ TECHNOLOGIST: ______________

GROUT MIXTURE & QUANTITY: ______________________________

GROOVE ORIENTATION AT TOP OF CASING: ______________________
(Draw diagram indicating groove orientation
reference point for slope indicator sensor.) ______________________

COMMENTS: (Installation Procedures Etc.) ______________________

**ALBERTA**
**TRANSPORTATION & UTILITIES**
MATERIALS ENGINEERING BR.
Geotechnical Services Section

## PIEZOMETER INSTALLATION RECORD

| | | | |
|---|---|---|---|
| PROJECT: | Hwy 16 & 41 – Proposed Interchange | PIEZOMETER NO. | No. 3–B |
| FILE NO. | B.F. 79440 | DATE INSTALLED: | Jan. 26, 1993 |
| LOCATION: | Sta 10 + 039.1 | SUPERVISOR | B. Wiens |
| | at 0.5m Lt Centerline | DRILLER | Mobile Augers & Research |
| | | TYPE OF DRILL | Track Mount B–61 |

**1. PIEZOMETER DETAILS**

| STANDPIPE | | PNEUMATIC | | OTHER |
|---|---|---|---|---|
| Type of Tip | Slotted | Trade Name | | Give Details Below: |
| Length of Tip | 9.1 meters | Model No. | | |
| Type of Pipe | 38 mm PVC | Tip No. | | |
| Total Length of Pipe Used | 30.5 meters | Length of Lead | | |

**2. INSTALLATION DETAILS**

Depth of Test Hole Drilled: 29.7 meters
Tip Installed in (Soil Type): Sand
Depth Tip Installed at: 20.6 m to 29.7 m
Stick–up for Standpipe Piezometer: 0.70 meters
Type of Backfill Material – From Bottom of Hole to Lower Seal or Filter Material:

| | | | | | |
|---|---|---|---|---|---|
| | | from | | to | |
| Type of Lower Seal Material | | from | | to | |
| Type of Filter Mat. About Tip | Native Sand | from | 18.6 m | to | 29.7 m |
| Type of Upper Seal Material | Bentonite Pellets | from | 18.0 m | to | 18.6 m |
| Type of Backfill Mat. Above Upper Seal | Native Soil cuttings | from | 0.30 m | to | 18.0 m |
| Type of Surface Seal Mat. | Bentonite Pellets | from | surface | to | 0.30 m |

**3. DETAILS OF ANY GROUTS USED FOR ABOVE MATERIAL** (TYPE, MIX PROPORTION, POSITION IN HOLE, etc.)

**4. TREATMENT OF STANDPIPE PIEZOMETER WITH ARTESIAN PRESSURE:**
Gauge Installed ______ Piezo Capped ______ Left Flowing ______

**5. COMMENTS ON INSTALLATION:**
(Difficulties Encountered, Reliability of Seals, Deviation From Normal Procedures, etc.)

Ground Surface Elevaton = 629.43 m

Top of Pipe Elevation = 630.13 m

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MATERIALS ENGINEERING BR.
Geotechnical Services Section

## PIEZOMETER INSTALLATION RECORD

| | | | |
|---|---|---|---|
| PROJECT: | Athabasca Bridge / North Side | PIEZOMETER NO. | 15829 |
| FILE NO. | BF 81129 | DATE INSTALLED: | Aug. 18, 1992 |
| LOCATION: | Sta. 35+083 | SUPERVISOR | B. Wiens |
| | 15.0 m Lt Centerline | DRILLER | D. Jennings |
| | | TYPE OF DRILL | Mayhew 1000 – Track |

### 1. PIEZOMETER DETAILS

| STANDPIPE | | PNEUMATIC | | OTHER |
|---|---|---|---|---|
| Type of Tip | | Trade Name | R.S.T. | Give Details Below: |
| Length of Tip | | Model No. | P – 104 | |
| Type of Pipe | | Tip No. | 15829 | |
| Total Length of Pipe Used | | Length of Lead | 46 m / 150 ft | |

### 2. INSTALLATION DETAILS

Depth of Test Hole Drilled: 10.4 meters
Tip Installed in (Soil Type): Clay Shale
Depth Tip Installed at: 9.80 meters
Stick–up for Standpipe Piezometer: N/A
Type of Backfill Material – From Bottom of Hole to Lower Seal or Filter Material:

| | | | | | |
|---|---|---|---|---|---|
| | | from | | to | |
| Type of Lower Seal Material | N/A | from | | to | |
| Type of Filter Mat. About Tip | Ottawa Sand – (saturated) | from | 9.2m | to | 10.4m |
| Type of Upper Seal Material | Bentonite Pellets | from | 8.2m | to | 9.20m |
| Type of Backfill Mat. Above Upper Seal | Grout | from | Surface | to | 8.20m |
| Type of Surface Seal Mat. | | from | | to | |

### 3. DETAILS OF ANY GROUTS USED FOR ABOVE MATERIAL (TYPE, MIX PROPORTION, POSITION IN HOLE, etc.)

Mix Proportions – 2 bags cement, 1/2 bag drill mud, and approx. 180 litres of water

### 4. TREATMENT OF STANDPIPE PIEZOMETER WITH ARTESIAN PRESSURE:

Gauge Installed ________ Piezo Capped ________ Left Flowing ________

### 5. COMMENTS ON INSTALLATION:

(Difficulties Encountered, Reliability of Seals, Deviation From Normal Procedures, etc.)

– Normal proceedures followed – No problems encountered.

– READINGS: – 1.8 p.s.i. @ completion of installation
– 1.1 p.s.i. on Aug. 19, 1992 – the following day

– NOTE: – Borehole was air drilled and completely dry at the time of installation

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## PIEZOMETER INSTALLATION RECORD

PROJECT: ______________________
FILE NO. ______________________
LOCATION: ______________________
______________________
______________________

PIEZOMETER NO. ______________
DATE INSTALLED: ______________
SUPERVISOR ______________
DRILLER ______________
TYPE OF DRILL ______________

### 1. PIEZOMETER DETAILS

**STANDPIPE**
Type of Tip ____________
Length of Tip ____________
Type of Pipe ____________
Total Length of Pipe Used ____________

**PNEUMATIC**
Trade Name ____________
Model No. ____________
Tip No. ____________
Length of Lead ____________

**OTHER**
Give Details Below:
______________
______________
______________
______________

### 2. INSTALLATION DETAILS

Depth of Test Hole Drilled: ______________
Tip Installed in (Soil Type): ______________
Depth Tip Installed at: ______________
Stick-up for Standpipe Piezometer: ______________
Type of Backfill Material – From Bottom of Hole to Lower Seal or Filter Material:

| | | from | | to | |
|---|---|---|---|---|---|
| | ______________ | from | ______ | to | ______ |
| Type of Lower Seal Material | ______________ | from | ______ | to | ______ |
| Type of Filter Mat. About Tip | ______________ | from | ______ | to | ______ |
| Type of Upper Seal Material | ______________ | from | ______ | to | ______ |
| Type of Backfill Mat. Above Upper Seal | ______________ | from | ______ | to | ______ |
| Type of Surface Seal Mat. | ______________ | from | ______ | to | ______ |

### 3. DETAILS OF ANY GROUTS USED FOR ABOVE MATERIAL (TYPE, MIX PROPORTION, POSITION IN HOLE, etc.)

______________________________________________
______________________________________________

### 4. TREATMENT OF STANDPIPE PIEZOMETER WITH ARTESIAN PRESSURE:

Gauge Installed ________ Piezo Capped ________ Left Flowing ________

### 5. COMMENTS ON INSTALLATION:

(Difficulties Encountered, Reliability of Seals, Deviation From Normal Procedures, etc.)

______________________________________________
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MATERIALS ENGINEERING BRANCH
GEOTECHNICAL SECTION

PROJECT ______________ DATE ______________

LOCATION ______________________________

TECHNICIAN ______________________________

PROJECT ENGINEER ______________________________

DRILLING CONTRACTOR ______________________________

DRILLER ______________________________

HELPER ______________________________

DRILL TYPE ______________________________

| | EXCELLENT | GOOD | FAIR | POOR |
|---|---|---|---|---|
| DRILLER | | | | |
| HELPER | | | | |
| CO-OPERATION | | | | |
| CONDITION OF EQUIPMENT | | | | |

DRILL CREW ARRIVE ON TIME ? YES ______ NO ______

DID THEY HAVE ALL THE REQUIRED EQUIPMENT ? YES ______ NO ______

DID ANY MEMBER OF DRILL CREW HAVE FIRST AID ? YES ______ NO ______

COMMENTS ______________________________

SUGGESTIONS FOR IMPROVEMENTS ______________________________

# ON-SITE MEETING AGENDA
# SAFETY ITEMS, CONCERNS AND CHECK-LIST

## 1. OCCUPATIONAL HEALTH AND SAFETY, AMENDED ACT, 1994

### 1.1 Requirements of "Prime Contractor" and "Contractor"

- Review responsibilities of Prime Contractor.
  Comments________________________________________
  ________________________________________________

- Advise Contractor and sub-contractor that he is responsible for health and safety of workers.
  Comments________________________________________
  ________________________________________________

- Review procedure (i.e., corrective action and eventual cessation of work) to handle "imminent danger" with Contractor.
  Comments________________________________________
  ________________________________________________

### 1.2 First Aid Regulations

- Are there First Aiders on site?
  Yes______No______Names__________________________

- Are there first aid supplies?
  Yes_______No_______Location______________________

- Is a means of communication to the Ambulance Service accessible to the worksite?.
  Yes______Type and Location______________________No_______
  No________Action________________________________

- Is Emergency Conveyance available at the worksite?
  Yes______Location_______________________________No______
  No________Action________________________________

### 1.3 General Safety Regulation

**General Provisions**

**Traffic Hazards**

- Is there a necessity for protection of workers from traffic by signs, barricades and/or flagpersons?
  Yes______Action_________________________________No______

- Is there a necessity of wearing safety vests?
  Yes______No_______Action________________________

- Are truckers wearing safety vests when on the road surface?
  Yes______No_______Action________________________

**Illumination**

- Worksite must be lit during darkness. Agreed__________

- Flagpersons station must be lit during darkness. Agreed___________

**Housekeeping**

- Work areas, platform, walkways must be free of debris & obstruction that would endanger workers. Agreed__________

**Overhead Powerlines**

- Are equipment and workers maintaining required distances from powerlines? Yes_______No_______Action____________________

- Have overhead powerlines been identified and the hazards conveyed to workers? Yes_______No_______Action____________________

**General Provisions Respecting Machinery**

**General Safety Requirements**

- Is equipment maintained to manufacturer's specifications? Yes_______No_______Action____________________

- All equipment maintenance is the responsibility of the owner/contractor. Agreed__________

**Personal Protective Equipment (P.P.E.)**

- Are workers provided with and trained in the use of proper P.P.E.? Yes_______No_______Action____________________

**Powered Mobile Equipment and Other Vehicles**

**Backup Alarm System - Automatic**

- Are construction vehicles and equipment equipped with automatic backup Alarm System? Yes_______No_______Action____________________
  Equipment Exemptions____________________

- Are there manual switches to activate or shut off the alarm? Yes_______No_______Action____________________

**If the Field Technologist/designate (Consultant Technologist) becomes aware of equipment working without a fully functional automatic backup alarm, that piece of equipment will be shut down until the backup alarm becomes fully functional.**

**Powered Mobile Equipment - ROPS**

- Is equipment equipped with roll-over-protective structures?
  Yes______No______Action________________________________________________

- Are seat belts used by equipment operators?
  Yes______No______Action________________________________________________

**Hoisting & Hoists**

**Vehicle Hoists**

- Does operator of vehicle hoist remain at controls while vehicle hoist is in motion?
  Yes______No______Action________________________________________________

**Winching Operations**

- Are Contractor and workers aware of safety involved with winching operations?
  Yes______No______Action________________________________________________

**Rigging**

- Are Contractor and workers aware of regulations regarding rigging operations?
  Yes______No______Action________________________________________________

**Excavations, Trenches**

**Excavations**

- Are workers and Contractor aware of safety requirements for working in excavations?
  Yes______No______Action________________________________________________

- Are workers and Contractor aware of safety requirements in shoring?
  Yes______No______Action________________________________________________

**Trenching**

- Are workers and Contractor aware of safety requirements to protect workers from cave-ins or sliding materials?
  Yes______No______Action________________________________________________

- Are workers and Contractor aware of safety requirements in shoring?
  Yes______No______Action________________________________________________

**Oil & Gas Servicing & Drilling**

**Inspection of Rigs**

- Has drill rig been inspected by a competent worker?
  Yes______When__________________________By whom?________________________________
  No______Action________________________________________________________________

**Raising, Lowering and General Operation of Derricks**

- Are all workers aware of the proper operation of derricks?
  Yes______Action________________________________________________________No______

- No worker slides down a pipe, kelly hose, cable or rope on a derrick.
  Agreed_________

**Auxiliary Escape**

- Is there a ready, convenient & safe alternate escape route?
  Yes______No______Action__________________________________________________

**Drilling Fluid Pumps**

- Ensure that drilling fluid pump and pressure relief device and the piping and hoses thereon and work in connection therewith comply with Section 202 of Regulations.
  Agreed_________

**Catheads**

- Ensure that a cathead and work in connection with a cathead comply with Section 203 of Regulations.
  Agreed_________

**Brakes**

- Ensure brakes on the drawworks of a drilling rig are tested at the beginning of each crew shift and are examined at least weekly to ensure they are in good working order.
  Agreed_________

**Racking Pipes in Derricks**

- Ensure that racking pipes in derricks comply with Section 212 of Regulations.
  Agreed_________

## 2. TRAFFIC ACCOMMODATION - PUBLIC SAFETY

### 2.1 Signing

- Do signs meet specifications?
  Yes______No______Action________________________________________

- Are signs erected before work commences?
  Yes______No______Action________________________________________

- Are signs inspected at regular intervals?
  Yes______No______Action________________________________________

### 2.2 Use of Specific Signs to Advise Motorist

- What specific signs are used to advise motorists?
  ________________________________________________________
  ________________________________________________________

### 2.3 Flagpersons

- Are flagpersons trained and certified?
  Yes______No______Action________________________________________

- Do they dress properly - white coveralls, orange hard hat, reflective safety vest?
  Yes______No______Action________________________________________

- Number of flagpersons.________________

- What kind of communications device is used between flagpersons?
  ________________________________________________________

## 3. Trucks

- Are trucks carrying unnecessary riders?
  Yes______No______Action________________________________________

- All trucks must comply C.V.S.A. regulations. Agreed________

## 4. Utilities and Railways

- Are all utilities staked and signed?
  Yes______No______Action________________________________________

- Are workers aware of Pipeline Act and Regulations?
  Yes______No______Action________________________________________

- Have railway authority been contacted if working near tracks or crossings?
  Yes______No______Action________________________________________

5. **W.H.M.I.S. and Transportation of Dangerous Goods**

- Are employees trained?
  Yes______Names______________________________________________No_______

6. **General**

- Is work done along school bus routes?
  Yes_______No_______

- Parking of construction equipment at night.
  Location______________________________________________

- Parking of employee private vehicles.
  Location______________________________________________

7. **Reporting of Accidents Within Project Limits**

- To the Field Technologist/Consultant Technologist.
  Date and Time______________________________________________

- To the Geotechnical Services Office (427-3101).
  Date and Time______________________________________________

- Report serious injuries to O.H.& S. Branch.
  Date and Time______________________________________________

8. **Monitoring of Drilling Projects**

- Name of Field Technologist/Consultant Technologist.______________________________

- Name of AT&U Geotechnical Engineer.______________________________

- Has copy of inspection report from AT&U Safety Officers and/or OH&S Officers been attached?
  Yes_______No_______Action______________________________

9. **Accident Reports**

- Report of powerline strikes to AOH&S, Alberta Labour and the Geotechnical Office.
  Date and Time______________________________________________

- Report of body injuries to RCMP, hospital and Geotechnical Office.
  Date and Time______________________________________________

- Report of equipment accidents to RCMP, drilling company and Geotechnical Office.
  Date and Time______________________________________________

10. **Employee Suggestions**

______________________________________________
______________________________________________
______________________________________________
______________________________________________
______________________________________________

# ON-SITE MEETING AGENDA
# SAFETY ITEMS, CONCERNS AND CHECK-LIST

Project No.:______________________ Date:______________________
Project Name:______________________
Project Location:______________________ Meeting Location:______________________
Company Name:______________________
Drilling Co.:______________________

| NAME | COMPANY | POSITION/SIGNATURE |
|---|---|---|
| 1. | | |
| 2. | | |
| 3. | | |
| 4. | | |

| TRANSPORTATION & UTILITIES REPRESENTATIVES/CONSULTANT TECHNOLOGIST | POSITION/SIGNATURE |
|---|---|
| 1. | |
| 2. | |
| 3. | |

Estimated Start-up Date: ______________________
Estimated Days of Work:______________________

EMERGENCY TELEPHONE NUMBERS

Transportation & Utilities Project Manager/Designate:______________________
AT&U Contact Person: ______________________
R.C.M.P. Detachment/Telephone No.:______________________
Hospital Location/Phone No.:______________________
Ambulance Location/Phone No.:______________________
Gas Utility:______________________
Power Utility:______________________
Report Faxed to Geotechnical Office (427-1265) on ______________________
Report Faxed to Drilling Co. on______________________

# C ROCK QUALITY DESIGNATION CLASSIFICATION OF ROCK PHOTOS OF CORE SAMPLES

## ROCK QUALITY DESIGNATION

The rock quality designation (RQD) is based on a modified core recovery procedure which, in turn, is based indirectly on the number of fractures and the amount of softening or alteration in the rock mass as observed in the rock cores from a drillhole. Instead of counting the fractures an indirect measure is obtained by summing up the total length of core recovered but counting only those pieces of core which are 4 in (10 cm) in length or longer, and which are hard and sound.

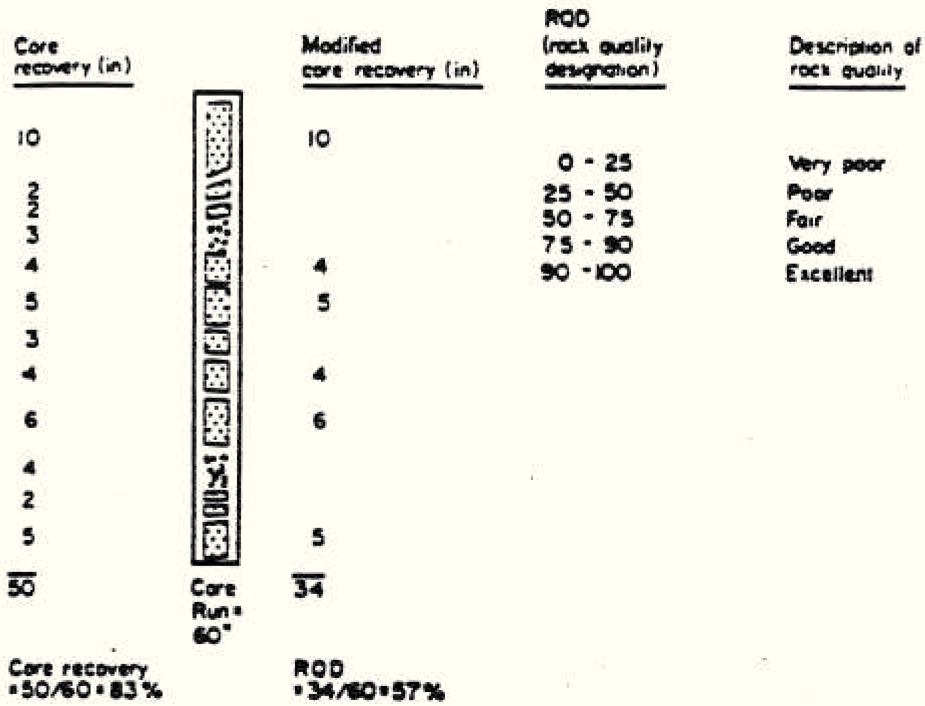



Figure 1 Modified core recovery as an index of rock quality

An example is given in figure 1 from a core run of 60 in. For this particular case the total core recovery was 50 in, yielding a core recovery of 83%. On the modified basis, only 34 in are counted and the RQD is 57%. It has been found that the RQD is a more sensitive and consistent indicator of general rock quality than is the gross core recovery percentage.
If the core is broken by handling or by the drilling process (i.e. the fracture surfaces are fresh irregular breaks rather than natural joint surfaces), the fresh broken pieces are fitted together and counted as one piece, provided that they form the requisite length of 4 in.

## CLASSIFICATION OF ROCK

| STRENGTH | | FIELD IDENTIFICATION METHOD | RANGE OF UNCONFINED COMPRESSIVE STRENGTH (MPa) |
|---|---|---|---|
| GRADE | CLASSIFICATION | | |
| RO | Extremely Weak | Indented by thumb nail | <1 |
| R1 | Very Weak | Crumbles under firm blows of geological hammer can be peeled with a pocket knife | 1 – 5 |
| R2 | Weak Rock | Can be peeled by pocket knife with difficulty; shallow indentations made by a firm blow with point of geological hammer | 5 – 25 |
| R3 | Medium Strong | Cannot be scraped or peeled with a pocket knife; Specimen can be fractured with a single firm blow of geological hammer | 25 – 50 |
| R4 | Strong | Specimen requires more than one blow geological hammer to fracture. | 50 – 100 |
| R5 | Very Strong | Specimen requires many blows of geological hammer to fracture | 100 – 250 |
| R6 | Extremely Strong | Specimen can only be chipped by the geological hammer | >250 |

Source : Canadian Foundation Engineering Manual, 3 rd Edition

CORE SAMPLE: RUN 5 (8.8 - 11.6 m)

CORE SAMPLE : RUN 3 (8.6 - 11.6 m)

DATE : Feb 12/ 93 TAKEN BY : G. Spicer FILE : BF 7066

GEOTECHNICAL SERVICES SECTION

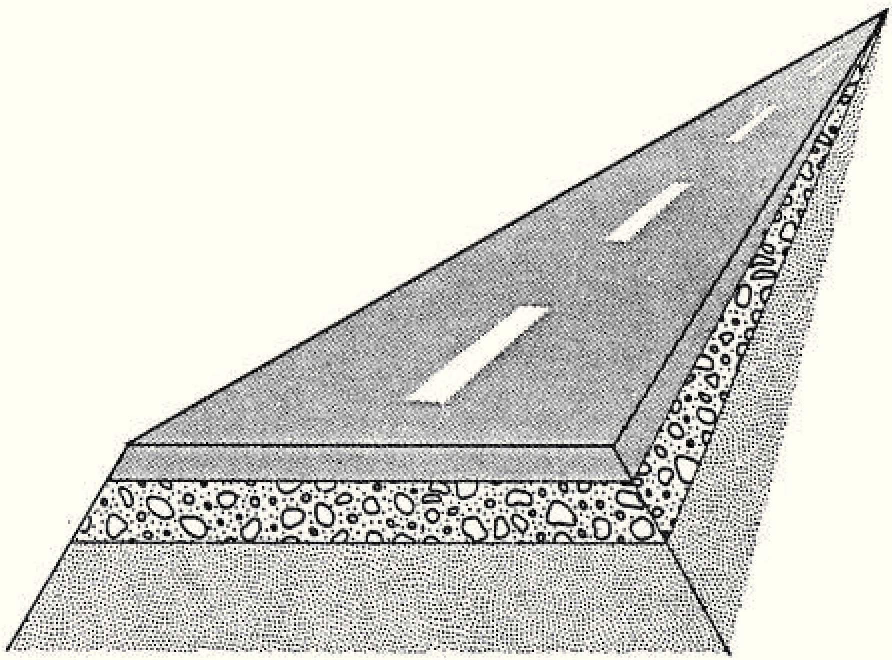
# D CHAPTER 4 INTRODUCTION TO SOILS
# CHAPTER 5 PRELIMINARY SOILS SURVEY

(AT&U Materials Manual, MEB 1, 1990)

Alberta
TRANSPORTATION

# MATERIALS MANUAL

# MEB 1



MATERIALS ENGINEERING BRANCH

90.04 EDITION

# 4.0 INTRODUCTION TO SOILS

This chapter outlines the various soil constituents, some of the engineering properties of soil, and moisture-density relationships.

In Civil Engineering, soil is defined as any earthen material, excluding bed rock, and is composed of loosely bound mineral grains of various sizes and shapes, organic material, water and gases.

## 4.1 Texture

The size and distribution of the mineral grains in a soil determine its texture.

Based on the grain size distribution, soils are divided into two broad categories:

a) Coarse grained soils, and

b) Fine grained soils.

### 4.1.01 Coarse-Grained Soils

Coarse-grained soils are soils mainly composed of particles greater than 0.08 millimetres (80 µm). This group has been divided into boulder, cobble, gravel and sand fractions.

Boulders are particles larger than 300 mm.

Cobbles range from 300 mm to 75 mm in size.

Gravel particles range from 75 mm to 5 mm (5 000 µm). According to particle size, gravels are further divided into coarse and fine fractions.

Sand particles range from 5 000 µm to 80 µm. Sands are further divided into coarse, medium and fine fractions.

The diagram below shows the relationship between the sizes of coarse grain soil types and fractions.

| COARSE GRAINED SOILS | | | | | | |
|---|---|---|---|---|---|---|
| Boulder | Cobble | Gravel | | Sand | | |
| | | Coarse | Fine | Coarse | Medium | Fine |

300 mm 75mm 20mm 5mm 2mm 400 µm 80 µm

### 4.1.02 Fine-Grained Soils

Fine-grained soils are soils mainly composed of particles smaller than 80 µm. This group is sub-divided into silts and clays and organic materials.

Silts are the result of mechanical weathering. Silts are mainly composed of particles smaller than 80 µm and larger than 2 µm.

Clays are mostly plate like mineral particles which are the result of chemical weathering. Clays are usually smaller than 2 µm, however, clay particles can be as large as silt particles.

Silts and clays are further divided into coarse, medium and fine fractions. The diagram below illustrates the relationship between the sizes of silt and clay and their fractions.

| FINE GRAINED SOILS | | | | | |
|---|---|---|---|---|---|
| SILT | | | CLAY | | |
| Coarse | Medium | Fine | Coarse | Medium | Fine |
| 80 µm | 20 µm | 6 µm | 2 µm | 0.6 µm | 0.2 µm |

Organic materials consist of either partly decomposed vegetation as in peat, or finely-divided vegetable matter as in organic silts and clays. These materials are not identified according to particle size as they are often fibrous in nature, and can usually be detected visually or by their distinctive odor.

### 4.1.03 Application

It is not sufficient to describe a soil as being coarse-grained, fine grained, or organic in nature. These designations primarily describe particle size and do not provide any explanation as to the properties of the soil itself, information which is of prime importance in Civil Engineering. Consequently, a more detailed system is used to describe a soil. This system classifies the sample based on its gradation and moisture characteristics.

## 4.2 Gradation

The properties of a soil are affected by the distribution of the soil particles. The relative amount of each size of soil particle present in a soil sample is called its grading or gradation. The gradation of coarse grained soils is determined by performing a sieve analysis on a representative sample.

The particle size distribution of fine grained soils can be determined by performing a hydrometer analysis.

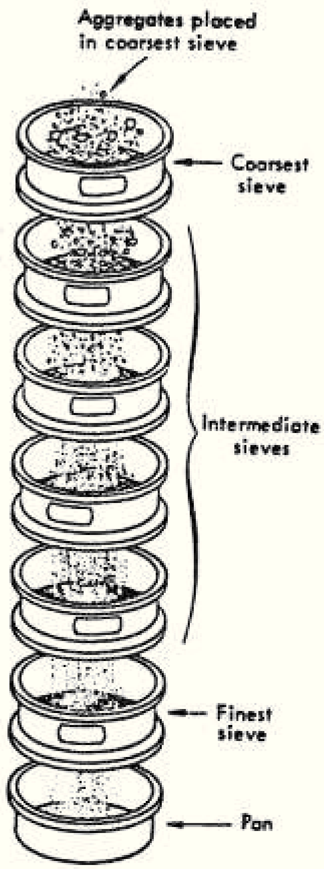



FIGURE 4.1

## 4.2.01 Sieve Analysis

In the procedure, the soil sample is washed, then dried and passed through various screens called sieves, as shown in Figure 4.1. The size of the openings in the screens is in micrometres, abbreviated µm.

The following sieves are available:

80 000, 50 000, 40 000, 25 000, 20 000, 16 000, 12 500, 10 000, 5 000, 2 500, 1 250, 630, 315, 160 and 80 µm. The required series of sieves larger than the 1 250 µm sieve is shown in the contract specifications and depend on the topsize of material and its intended use. All of the sieves from the 1250 µm to 80 µm should be used. The -1250 to 80 µm data provides a history of the aggregate source.

From the test, the weight of material retained on each sieve is used to calculate the cumulative weight passing the sieve. The cumulative percent passing each sieve is calculated by comparing the cumulative weight passing the sieve to the original sample dry weight.

The result (grading) indicates the distribution of particle sizes in the soil which is of significance for engineering purposes. Since gradation control of coarse grained soils is only exercised in the field during the production of the aggregate, its importance is discussed in Chapter 8.0, AGGREGATES.

## 4.2.02 Hydrometer Analysis

The hydrometer analysis test determines the distribution of particle sizes smaller that 80 µm. The method uses a hydrometer to measure the density of water and suspended soil at specific time intervals and is based on the rate of sedimentation of various soil particles.

Before performing the hydromter analysis test, the sieve analysis and Atterberg Limits (Section 4.3.04) of the soil are determined. The fine soil classification (Section 4.4) is used to determine the amount of -2000 µm air dried soil required for the hydrometer analysis test.

A moisture content test (Section 4.3.01) is performed on the -2000 µm air dried soil. This value is used to correct for hygroscopic moisture the hydrometer analysis test sample weight.

The relative density of the soil (Section 4.7.02) is determined. This value is used for the hydrometer analysis test calculations.

In the hydrometer test, 50 to 80 g of air dried -2000 µm soil is placed in a 50 ml beaker and covered with 125 ml of distilled water and dispersing agent solution. The mixture is washed into a blender where it is stirred for 1 minute. The soil-water slurry is immediately transferred to the test cylinder and distilled water is added to the 1 litre mark. The mixture is allowed to sit undisturbed for at least 16 hours.

A rubber stopper is put on the open end and the cylinder is turned upside down and back for 1 minute (1 turn/s). The cylinder is set down and hydrometer and temperature readings are taken at 15 s, 30 s, 1 min, 2 min, 5 min, 15 min, 30 min, 60 min, 250 minutes and 24 hours. After the final reading, the suspension is transferred to an 80 µm sieve and washed. The +80 µm material is dried and sieved on the 1250, 630, 315, 160 and 80 µm sieves.

Hydrometers are graduated to read correctly at the surface of the liquid, that is, the bottom of the meniscus. Since the water is too dark, readings are taken at the top of the meniscus and then corrected. The **actual** hydrometer reading is determined by adding the meniscus correction factor to the reading taken.

The diameter (D) of a particle corresponding to the percentage given by a hydrometer reading is calculated as follows:

$$D = K\sqrt{\frac{L}{T}}$$ where:

K = Constant which depends on the temperature of the suspension and the relative density of the soil.

L = Distance from the surface of the suspension to the level of the actual hydrometer reading.

T = Time interval in min from beginning of sedimentation to time reading taken.

The percent (P) of material finer than the average particle diameter is calculated using the formula:

$$P = \frac{Drs}{W\ (Drs - 1.000)} \times R \times 100\%$$ where:

W = Weight of test sample corrected for moisture

Drs = Relative Density of the soil solids

R = Meniscus corrected hydrometer reading, further corrected for:

a) Dispersing agent, which increases the relative density of distilled water.

b) Temperature, which produces inaccuracies in the actual hydrometer readings when it deviates from the calibration temperature of 20°C.

The average "particle size" is plotted horizontally in a semi-log scale versus the "percent finer than" vertically using a linear scale (as for sieve analysis).

## 4.3 Moisture Characteristics

Particle size distribution does not identify the properties of the soil. The amount of water a soil can hold, and the way a soil behaves with varying amounts of water in its structure are important properties in civil engineering.

### 4.3.01 Moisture Content

Water content (moisture content) of soils in civil engineering is expressed as a percent and is calculated using the following formula:

$$\text{Moisture Content (\%)} = \frac{\text{Weight of Water Removed}}{\text{Weight of Dry Soil Solids}} \times 100\%$$

In the moisture content test procedure (ATT-15), a representative sample of the moist soil is weighed in a container. The sample is placed in an oven and dried to a constant weight at a temperature of 110°C, ±5°C.

The sample dry weight (plus the container) is subtracted from the original wet weight (plus the container) to determine the weight of water removed. The weight of the container is subtracted from the weight of the dry soil and container to determine the weight of the dry soil solids. The obtained values are substituted in the formula and the moisture content is calculated.

### 4.3.02 Moisture Capacity

Water in a soil distributes as a very thin film about each soil particle. Consequently, the larger the composite surface area of a soil, the higher the moisture capacity. If one large stone could be pulverized, the total surface area would be greatly increased. Therefore, the smaller the composite particle in a given soil sample, the larger the total surface area, and the higher the moisture capacity. Clays and silts are the smallest of the soil particles and have a much higher moisture capacity than sands and gravels.

### 4.3.03 States of Soil

The condition of a soil can be altered by changing the moisture content; for example, clay softens whenever water is added. Most soils can exist in four basic conditions depending on their moisture contents. They are:

1. the dry (powder) state,
2. the semi-solid state,
3. the plastic state, and
4. the liquid state.

In the dry or **powder state** the soil contains little or no moisture and is cohesionless.

If increasing amounts of water are added to the soil in the powder state, the soil will enter the **semi-solid state.** In this condition the soil appears dry. If the particles are held together to form a lump, the lump will crumble under pressure.

By adding increasing amounts of moisture to a soil in its semi-solid state, the soil will enter into the **plastic state.** The particles become lubricated and the material can be easily molded and maintains a molded shape.

Further increases in moisture content to a soil in the plastic state, will cause the soil to enter into the **liquid state.** In this condition the film of moisture coating each soil particle becomes sufficiently thick that the particles become suspended in water, giving the soil a fluid consistency.

### 4.3.04 Atterberg Limits

For every fine grained soil there is a range of moisture contents within which the clay is of a plastic consistency. The Atterberg Limits provide a way of measuring and describing the plasticity range in numerical terms.

The range of moisture contents through which a soil remains in the plastic state is part of a soils identification. The plastic state of a soil begins at the transition from the semi-solid state to the plastic state, and ends at the transition from the plastic state to the liquid state. Thus, the range of moisture contents through which a soil exists in the plastic state is found by determining the moisture content at the two transition points. These two moisture contents comprise the Atterberg Limits of a soil and are called the Plastic Limit and the Liquid Limit. The range of the plastic state, the numerical difference between the two limits, is called the Plasticity Index.

The Atterberg Limits of a soil are illustrated in Figure 4.2.

**RELATIONSHIP OF ATTERBERG LIMITS TO STATES OF SOIL**

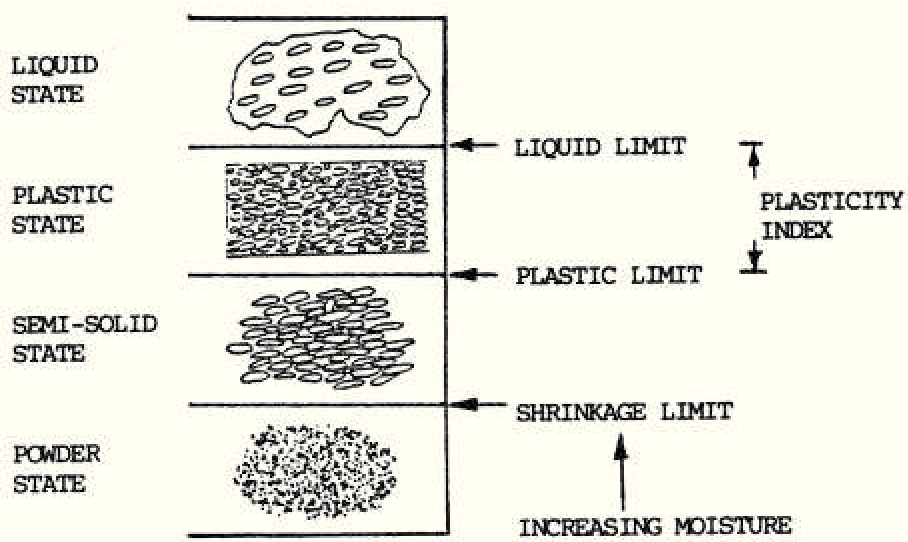



**FIGURE 4.2**

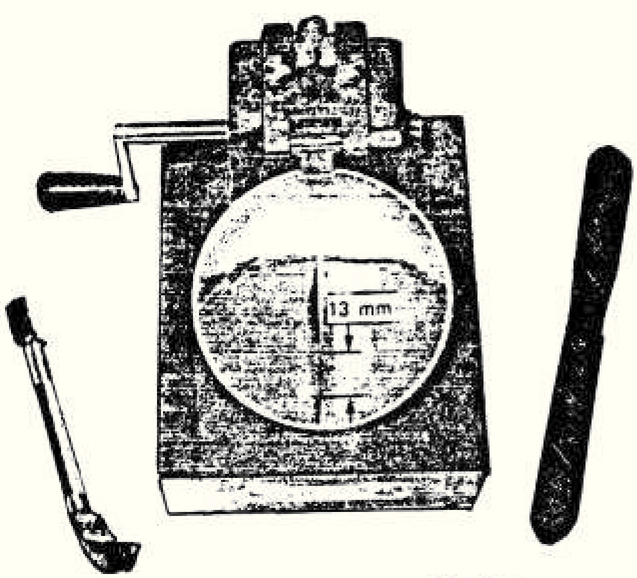
The change from one phase to the next is not observable as a precise boundary, but takes place as a gradual transition. Nevertheless, arbitrary but specific boundaries have been established. The moisture contents at these boundaries are known as the liquid limit, plastic limit, and shrinkage limit.

### 4.3.04.1 Liquid Limit

The moisture content at the transition between the plastic and liquid states is called the Liquid Limit (LL). It is defined as the moisture content at which the bottom of the groove in a pat of soil in the liquid limit machine will close for a distance of 13 mm when given 25 blows.

The procedure for determining the Liquid Limit is as follows:

The soil sample is dried. A representative portion of the sample is razzed to break down the clay and silt lumps. The razzed material is sieved through the 400 μm sieve. The material which passed the 400 μm sieve is wetted (with distilled water) to the estimated moisture condition of the soil's liquid limit. The sample and water are thoroughly mixed. The wet -400 μm material is left undisturbed until the water is uniformly distributed throughout the soil (usually overnight). While sitting, the sample is protected from evaporation.




Liquid limit device with soil sample in place.

FIGURE 4.3

The sample is then re-mixed, a portion is spread on the lower end of the cup in the liquid limit device and levelled off (covering between 1/2 and 2/3 of the cup. The excess soil is returned to the mixing dish. A "V" shaped groove is made with the grooving tool along the diameter of the cup, through the center of the soil and separating it, as shown in Figure 4.3. The cup containing the sample is then lifted and dropped at a rate of 2 rev/s, until the two sides of the sample contact the bottom of the groove for a distance of 13 mm. The number of blows required to close the groove for a distance of 13 mm is recorded.

If the number of blows given to the sample is between 20 and 30, one moisture content sample is taken from each side of the groove. Each moisture sample is weighed in a tared container and oven dried at 110°C ±5°C to constant weight.

The average moisture content is converted to the moisture content corresponding to 25 blows. This is the Liquid Limit of the soil. For the conversion, the Department's Laboratory developed a curve of % Moisture versus number of blows for each significant soil type.

If the number of blows given to the sample is less than 20, the sample is too dry. In this case, more water is added and the run is repeated. If the sample took more than 30 blows to close the groove, the sample is dried and the procedure is repeated.

#### 4.3.04.2 Plastic Limit

The Plastic Limit (PL) is the moisture content at the transition between the semi-solid state and the plastic state. It is defined as the lowest moisture content at which the soil can be rolled into a thread of 3 mm diameter without the thread breaking into pieces.

The procedure for determining the plastic limit is as follows:

If both the liquid limit and plastic limit are required, the soil prepared for the LL test and remaining in the mixing dish is air dried to slightly above the estimated PL. The soil is shaped into a cylinder and then placed vertically on a table as to expose the most surface area. To speed up the drying process, an electric heater and fan combination blow hot air across the samples. The samples must not be allowed to over dry. Crusted portions must be removed as as they will not re-absorb water.

If only the plastic limit is required, a sample of -400 µm dried and razzed material is wet to slightly above the estimated PL. The sample is allowed to sit undisturbed and protected from evaporation until the water is uniformly distributed throughout the sample.

The moist soil is shaped into a ball. The soil ball is lightly squeezed and then rolled between the fingers and a ground-glass plate set on an horizontal surface. The mass is rolled into a thread of uniform diameter until it reaches a diameter of 3 mm. Then the thread is broken into pieces, the pieces are squeezed together and the mass is re-rolled to 3 mm in diameter. The rolling, breaking off, kneading and re-rolling is continued until the 3 mm diameter thread barely holds together, that is, the thread would crumble if rolled once more.

The threads are weighed in a tared container and oven dried at 110°C to constant weight. The moisture content of the 3 mm worms is the Plastic Limit of the soil.

#### 4.3.04.3 Plasticity Index

The numerical difference between the Liquid Limit and the Plastic Limit is called the Plasticity Index (abbreviated "PI"). This represents the range of moistures that the soil is in the plastic state.

Plasticity Index = Liquid Limit - Plastic Limit

Clay has the smallest particle size and the largest surface area per unit volume. As a result, clay remains in the plastic state through a wide moisture range and possesses the highest Plasticity Index. The quantity and type of clay in a soil affects that soil's Plasticity Index. For most soils, the higher the clay content, the higher the PI. Following are typical PI's for known clay contents.

| | | |
|---|---|---|
| 10% clay | - | PI = 6 |
| 25% clay | - | PI = 21 |
| 40% clay | - | PI = 72 |

Soils with a PI of 27% or more are high plasticity soils, those between 15% and 27% are medium plasticity soils, and those below 15% are low plasticity soils.

### 4.3.05 Plasticity Chart

The plasticity chart, as shown in Figure 4.4, describes soil types based on their Plasticity Index and Liquid Limit. In other words, the chart describes the characteristics that the soil displays with varying amounts of water in its structure. Divisions between the various soil types have been established at specific moisture contents for the Liquid Limit, horizontally, and for a definite range of plasticity (PI), vertically.

To determine the classification of a soil, the Liquid Limit is plotted against the Plasticity Index on the chart. The soil is labelled with the group symbols pertaining to the area of the chart in which it plots.

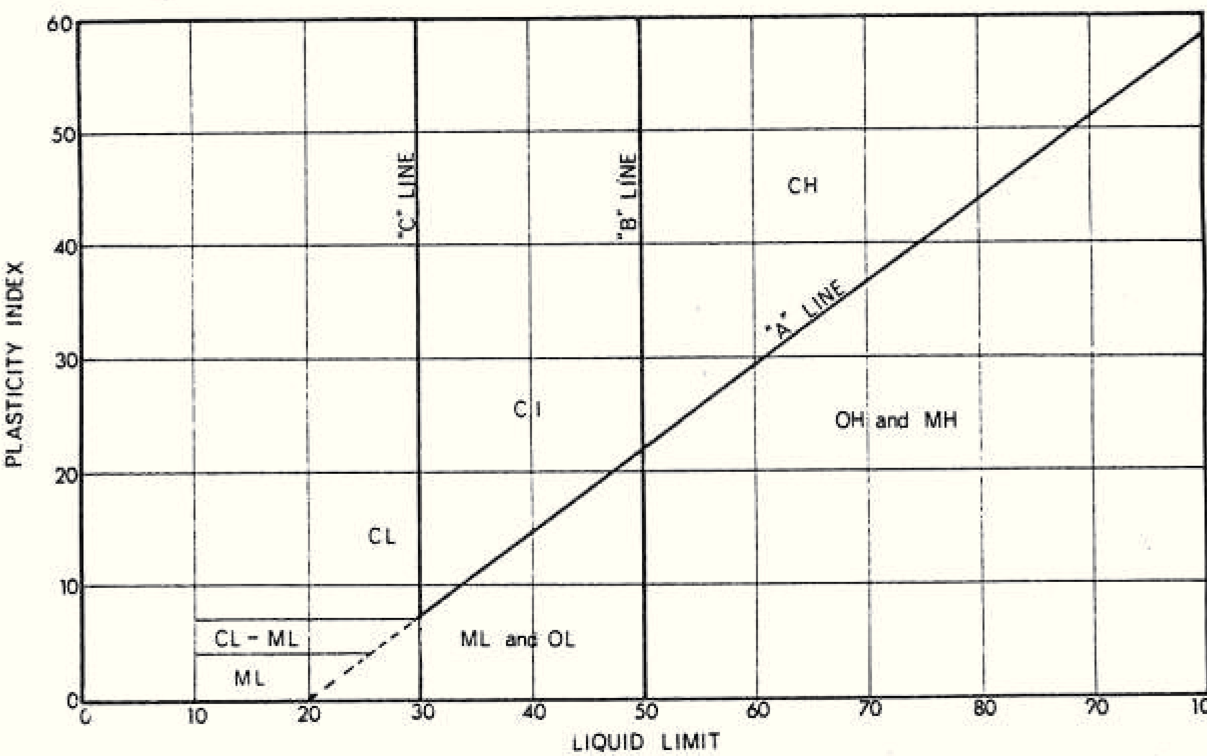



FIGURE 4.4

### 4.3.06 Liquidity Index

The Liquidity Index of a soil is the relationship of the natural moisture content of a soil to moisture contents of that soil at both its Plastic and Liquid Limits. It may be described as how close the natural moisture content is to these limits. The Liquidity Index equals 0.0 at the Plastic Limit and 1.0 at the Liquid Limit. It is negative below the Plastic Limit (below optimum) and is over 1.0 when the soil is in a fully liquid state. The formula is:

$$\text{Liquidity Index} = \frac{\text{Natural Moisture Content} - \text{Plastic Limit}}{\text{Plasticity Index}}$$

In carrying out a test excavation in highly plastic clay for the Winnipeg floodway, the Federal Prairie Farm Rehabilitation Administration (P.F.R.A.) found that rubber tired equipment had little trouble excavating if the Liquidity Index was below about 0.20 but excavation became increasingly more difficult as the Liquidity Index increased. In the soil at the Floodway, they had to use draglines when the Liquidity Index exceeded about 0.45.

Preliminary soil survey test results summary sheets, as shown in Figure 4.5, include the Liquidity Index for each soil sample. The Liquidity Index may be used to anticipate excavation difficulties in deep cuts. It also may be used to predict practical depth limits for borrow pits, considering that the moisture content of a soil does not vary appreciably over long periods of time below a depth of about 3.5 m.

### 4.3.07 Estimated Cohesiveness Using Dry Strength

The relative degree of cohesiveness of non-plastic soils is determined by relating it to the dry strength of the soil.

The field procedure (ATT-54) is performed on the -315 µm fraction of pitrun aggregates. The Department's Laboratory procedure is performed on the -400 µm fraction of a soil which is estimated as non-plastic. In the procedure, a sample of approximately 60 g of -315 or -400 µm dried soil is progressively wetted until the soil is easily formed into a ball. The ball is then oven dried.

Pressure is applied to the dried ball with the thumb and forefinger. The relative degree of cohesiveness of the soil is estimated as friable, low or medium, based on the pressure required to crumble the ball. If a solid hit with a hammer is required to crumble the ball, a high dry strength is reported.

The estimates are not related to the plasticity index. The PI of the non-plastic aggregate equivalent to the dry strength estimation is as follows:

## 4.3.07 Estimated Cohesiveness Using Dry Strength

| ESTIMATED DRY STRENGTH | EQUIVALENT PLASTICITY INDEX | |
|---|---|---|
| | RANGE | AVERAGE |
| friable (F) | 0 | 0 |
| low (L) | 0.5-1.5 | 1.0 |
| medium (M) | 1.0-2.0 | 1.5 |
| high (H) | 1.5-2.5 | 2.0 |

The Laboratory dry strength (DS) results are summarized in the Soil Survey Test Results sheet, as shown in Figure 4.5.

```
TSCS310C                              SOIL SURVEY TEST RESULTS        C. . . 1989

ALBERTA TRANSPORTATION                PROJECT NO : HW16 :04                    CC.  V. DIVALJEE, W. PARITI      (MEB)
MATERIALS ENGINEERING BRANCH          FROM     : E. OF MARLBORO                     R. OBERG                   (DIST. 08)
TESTING SERVICES SECTION              TO       : W. OF JCT HW47                     T. GEE                     (DEB)
DATE REPORTED = 89/10/25              PROJECT MANAGER        : P. ING          DATE SAMPLED   : 890900
REPORTED TIME = 15:30:53:49           MATERIALS TECHNOLOGIST : H. HAWLEY       ACCOUNT NUMBER : P707P

       SAMPLE TRANSIT                                                   EST. EST.
LAB    DEPTH  DEPTH   % PASSING SIEVE SIZE  LIQ.  PLA.  PLA.  FIELD LIQUID OPT. MAXIMUM SOIL           FIELD                  LABORATORY VISUAL
NO.    (M)    (M)     5000 1250 400  160  80 LIM.  LIM.  INDEX M.C.  INDEX  M.C. DENSITY CLASS *       REMARKS                  DESCRIPTION

 HOLE # 1        STATION : 53+820         LOCATION : 255mltclmed                        TOP SOIL : 0.30

448612 1.30   1.60     98   96   93 38.4 25.8  21.5  17.5   4.0  13.4 -1.02  14  1790   SMd-SC                           silty sand, clay lumps
448613 1.50            56   55   55 33.6 25.5  23.4  15.9   7.5  12.2 -0.48  13  1850   GC        st = 4.00 m            clayey sandy gravel

 HOLE # 2        STATION : 53+974         LOCATION : 45m ltclmed                        TOP SOIL : 0.15

448614 0.50   1.00    100   99   95 48.3 31.2 NP DS M           12.9  N/A   N/A  N/A    SMd                              silty sand, roots
448615 2.00   3.30     78   77   76 34.4 26.3 NP DS M           18.4  N/A   N/A  N/A    SMd       wt = 1.50 m            gravelly silty sand
448616 2.70            93        93      68.2  35.9  15.4  20.5  19.8 00.31  15  1820   CI        st = 3.00m             sandy clay, pebs

 HOLE # 3        STATION : 54+158         LOCATION : 36.5mltclme                        TOP SOIL : 0.20

448617 2.00   2.80    100  100   92 36.3 19.4 NP DS M           10.0  N/A   N/A  N/A    SMd                              silty sand, pebs
448618 3.00           100        99      90.4  35.7  21.9  13.8  30.1 00.59  22  1580   CI        st = 4.00 m            silty clay, sand lens

 HOLE # 4        STATION : 54+285         LOCATION : 39.5mltclme                        TOP SOIL : 0.15

448619 0.60   1.00     89   89   86 52.0 43.5  24.5  15.0   8.5  15.8 00.08  12  1890   SC                               clayey sand
448620 1.80   2.50     46   41   38 22.1 16.4  18.8  16.4   2.4   5.9 -4.37  12  1850   GMd                              gravelly sand
448621 3.40            48   41   38 19.1 15.1  20.7  16.2   4.5   6.4 -2.17  12  1850   GMd-GC    st = 4.20 m            sandy clayey gravel

 HOLE # 5        STATION : 54+420         LOCATION : 30mltclmed                         TOP SOIL : 0.30

448622 0.60   1.30    100  100   90 24.9 13.8 NP DS M            7.5  N/A   N/A  N/A    SMd                              silty sand
448623 1.80   3.05    100       100      93.7  39.3  19.9  19.4  24.8 00.25  20  1650   CI                               silty clay
448624 4.00           100        99      86.9  33.5  16.9  16.6  23.1 00.37  16  1770   CI        st = 4.50 m            silty clay

 HOLE # 6        STATION : 54+540         LOCATION : 24m ltclmed                        TOP SOIL : 0.15

448625 1.40   3.00    100  100   91 28.6 15.1 NP DS M           10.1  N/A   N/A  N/A    SMd                              silty sand
448626 3.30            97        97      87.7  36.7  21.5  15.2  25.5 00.26  21  1590   CI        st = 4.00 m            silty clay

 HOLE # 7        STATION : 54+640         LOCATION : 29.5mltclme                        TOP SOIL : 0.10

448627 1.00           100  100   87 32.0 15.5 NP DS M           10.3  N/A   N/A  N/A    SMd       st = 2.50 m/st = 3.00 m   silty sand

 HOLE # 8        STATION : 54+715         LOCATION : 6m rtcl med                        TOP SOIL : 1.80

448628 2.20           100        92      51.2  19.5  15.7   3.8  20.1 01.16  12  1890   ML        mu = 1.80m/wt = 3.0m/st = 3.5m clayey sandy silt

 HOLE # 9        STATION : 54+991.5       LOCATION : 30mltclmed                         TOP SOIL : 0.25

448629 1.00   1.40    100   98   89 37.4 14.6 NP DS M            8.7  N/A   N/A  N/A    SMd                              silty sand
448630 2.00           100   99   83 35.1 23.9 NP DS M           11.3  N/A   N/A  N/A    SMd       ro = 1.40 m/ st = 2.50 m   silty sand, sandstone
```

FIGURE 4.5

## 4.3.08 Estimated Maximum Dry Density and Optimum Moisture Content

The Department's Laboratory correlated the Atterberg Limits of fine grained soils to the soils' maximum dry density and optimum moisture content (Section 4.7.01). Six hundred soils with varying composition, plasticity and classification were used. Soils with standard 5-point moisture-density relation tests were selected.

The plasticity index of each soil was plotted vertically versus the liquid limit of the soil, horizontally. Each point was labelled with the soil's maximum dry density. Points of equal density were joined to form a straight line. The estimated maximum dry density of a soil could then be picked off the graph by plotting on it the known LL and PI of the soil.

Another graph of PI (vertically) versus LL (horizontally) was also plotted for each soil and it included the optimum moisture content of the soil. Points of equal optimum moisture content were joined. The resulting straight lines were used to estimate the optimum moisture content of a soil of known LL and PI.

Interpolation was required when the plotted point did not fall on a moisture or density line.

The data used to plot the curves has been entered into the computer. Once the LL and PI are entered, the computer estimates the optimum moisture content and maximum dry density of the soil. These values are printed out on the Soil Survey Test Results summary sheet, as shown in Figure 4.5.

Estimations are not performed for silts because silts are not recommended for road construction.

## 4.4 Unified Soil Classification System (Modified by PFRA)

For Laboratory soils identification, the Department uses the Unified Soil Classification System as modified by the Prairie Farm Rehabilitation Administration . A summary of the classification system is shown in Figure 4.6.

The gradation and moisture characteristics of the soil are used to classify soil samples. Sieve analysis and Atterberg Limits tests are routinely performed on soil samples received at the Department's Laboratory.

Based on the grain size distribution, soils are divided into coarse grained soils, and fine grained soils. Soils with less than 50% passing the 80 µm sieve are classed as coarse grained soils. Soils containing more than 50% passing the 80 µm sieve are fine-grained soils.

### 4.4.01 Coarse-Grained Soils

Coarse-grained soils are soils mainly composed of particles greater than 0.08 millimetres (80 µm). This group has been divided into boulder, cobble, gravel and sand fractions. Gravels and sands áre used for road construction.

#### 4.4.01.1 Soil Classification

Gravels are coarse grained soils having more than 50% of the sample's coarse fraction larger than the 5 000 µm sieve. That is, the majority of the sample's +80 µm material is retained on the 5 000 µm sieve.

Sands are coarse grained soils which have more than half of the sample's +80 µm material passing the 5 000 µm sieve.

Clean gravels or sands have little or no fines, that is, they contain less than 5% passing the 80 µm sieve. Dirty coarse grained soils contain an appreciable amount of fines, that is, more than 12% passing the 80 µm sieve.

#### 4.4.01.2 Group Symbols

Two letter symbols are used to identify and label coarse grained soils. The **first** letter represents the soil **type,** e.g., "G" for gravel, "S" for sand.

The **second** letter represents the particle size distribution or gradation of the **clean** aggregate. "W" stands for well graded and is used if the gravel or sand contains a good distribution of all sizes of soil particles. "P" represents a poorly graded aggregate. A poorly graded gravel or sand contains all sizes of soil particles but an excess of either coarse or fine particles. "P" is also used if the aggregate is uniformly graded, that is, the gravel or sand contains mostly one sized particles.

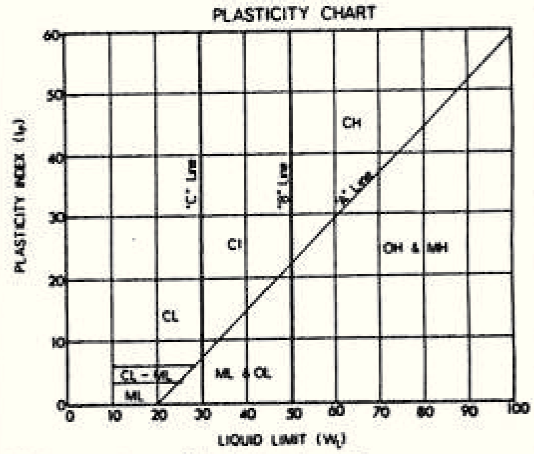
Alberta TRANSPORTATION

# UNIFIED SOIL CLASSIFICATION SYSTEM
## (Modified by PFRA)

| MAJOR DIVISIONS | | | GROUP SYMBOLS | LOG SYMBOLS | TYPICAL NAMES | LABORATORY CLASSIFICATION CRITERIA |
|---|---|---|---|---|---|---|
| COARSE GRAINED SOILS (More than half of material is larger than No. 80 sieve size) | GRAVELS (More than half of coarse fraction is larger than No. 5 000 sieve size) | CLEAN GRAVELS (Little or no fines) | GW | | Well - graded gravels, gravel - sand mixtures, little or no fines | $C_u = \frac{D_{60}}{D_{10}}$ is greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ is between 1 and 3 |
| | | | GP | | Poorly graded gravels, gravel-sand mixtures, little or no fines | Not meeting all gradation requirements for GW |
| | | GRAVELS WITH FINES (Appreciable amount of fines) | GM* d / u | | Silty gravels, gravel - sand - silt mixtures | Atterberg Limits below "A" Line or P.I. is less than 4 — Above "A" Line with PI between 4 and 7 are borderline cases requiring use of dual symbols |
| | | | GC | | Clayey gravels, gravel - sand - clay mixtures | Atterberg Limits above "A" Line with P.I. greater than 7 |
| | SANDS (More than half of coarse fraction is smaller than No. 5000 sieve size) | CLEAN SANDS (Little or no fines) | SW | | Well - graded sands, gravelly sands, little or no fines | $C_u = \frac{D_{60}}{D_{10}}$ is greater than 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ is between 1 & 3 |
| | | | SP | | Poorly graded sands, gravelly sands, little or no fines | Not meeting all gradations requirements for SW |
| | | SANDS WITH FINES (Appreciable amount of fines) | SM* d / u | | Silty sands; sand - silt mixtures | Atterberg Limits below "A" Line or P.I. less than 4 — Limits plotting in hatched zone with P.I. between 4 and 7 are borderline cases requiring use of dual symbols |
| | | | SC | | Clayey sands; sand - clay mixtures | Atterberg Limits above "A" Line with P.I. greater than 7 |
| FINE GRAINED SOILS (More than half of material is smaller than No. 80 sieve size) | CLAYS (Above "A" Line on PLASTICITY CHART; negligible organic content) | $W_L$ < 30% | CL | | Inorganic clays of low plasticity, gravelly clays, sandy clays, silty clays, lean clays | |
| | | 30 < $W_L$ < 50 % | CI | | Inorganic clays of medium plasticity, gravelly clays, sandy clays, silty clays | |
| | | $W_L$ > 50 % | CH | | Inorganic clays of high plasticity, fat clays | |
| | SILTS (Below "A" Line; negligible organic content) | $W_L$ < 50 % | ML | | Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity | |
| | | $W_L$ > 50 % | MH | | Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts | |
| | ORGANIC SILTS AND CLAYS (Below "A" Line) | $W_L$ < 50 % | OL | | Organic silts and organic silty clays of low plasticity | |
| | | $W_L$ > 50 % | OH | | Organic clays of medium to high plasticity, organic silts | |
| HIGHLY ORGANIC SOILS | | | Pt | | Peat and other highly organic soils | Strong colour or odor and fibrous textures |

Determine amount of sand and gravel from grained size curve. Depending on percent of fines (fraction smaller than No. 80 sieve size), coarse-grained soils are classified as follows:
Less than 5% ........ GW, GP, SW, SP
More than 12% ........ GM, GC, SM, SC
5 to 12% ........ Borderline cases requiring dual symbols**

PLASTICITY CHART




(NOTE For soils passing no. 400 sieve & sieve size in µm)

MAT 6-19/81

OTHER SYMBOLS

- BEDROCK (Unclassified)
- SANDSTONE
- SHALE
- LIMESTONE
- CONGLOMERATE
- COAL
- OVERBURDEN
- TOPSOIL

NOTE

* Division of GM and SM groups into subdivisons of d and u are for roads and airfields only. Subdivison is based on Atterberg Limits; suffix d is used when L.L. is 28 or less and the P.I. is 6 or less; the suffix u is used when L.L. is greater than 28.

** Borderline classifications, used for soils processing characteristics of two groups, are designated by combination of group symbols. For example: GW-GC, well-graded gravel-sand mixture with clay binder.

FIGURE 4.6

For gravels or sands containing an excessive amount of fines, the second letter represents the predominant fine soil in the sample, e.g., "M" for silt or "C" for clay.

Most soil samples contain varying amounts of all four soil types, but the soil is initially classified according to the predominant soil type in the sample.

If the coarse grained soil has between 5% and 12% passing the 80 µm sieve, the soil classification requires dual symbols: one symbol as for a clean coarse aggregate and one as for a dirty aggregate, e.g., GP-GM.

### 4.4.02 Fine-Grained Soils

Fine-grained soils are soils mainly composed of particles smaller than 80 µm. Therefore if more than 50% of the sample passes the 80 µm sieve, the soil is classified as fine grained. This group has been sub-divided into silts and clays and organic materials.

#### 4.4.02.1 Typical Soil

A typical fine grained soil, composed of approximately 25% sand, 50% silt, and 25% clay would possess a LL of 39%. The normal range is from 30% to 60%. The same soil would possess a PL of 17% with the normal range being from 12% to 25%.

This typical soil is composed of a combination of soil types. Each of the component soils have individual engineering characteristics which are used to quickly identify and classify the individual soil. However, when two or more individual soils are mixed to form a composite soil, its qualities vary with the proportions of its component soils. Consequently, a classification system must be able to distinguish these mixed soils and label them according to their characteristics. Most combination soils have liquid limits from 30% to 60%, and the typical soil represents the mean of the moisture properties for soils within this range.

#### 4.4.02.2 Soil Classification

To classify a fine grained soil sample, plot on the plasticity chart the plasticity index versus the liquid limit obtained on a portion of the -400 µm fraction of the sample. Pick off the chart the soil classification.

#### 4.4.02.3 Group Symbols

A two letter symbol is used to identify and label fine grained soils. The first letter is the symbol identifying the soil type i.e.,"M" for silt or "C" for clay. The second letter describes the water holding capacity or the plasticity or the compressibility of the soil, i.e., "H" for high, "I" for intermediate, and "L" for low.

A dual classification is given to soils that plot within ±2% of the moisture content of the "B" or "C" line on the plasticity chart. For example, a soil with a LL of 29% and a PI of 20% would be called a CL:CI, as it would display some characteristics associated with both materials.

#### 4.4.02.4 Organic Materials

Where organic material is detected in a soil, i.e., fibrous structure, distinctive odor, sponginess at the plastic limit, the material is given the symbol "O".

The Atterberg Limits of the organic soil are determined and the classification of "OL" or "OH" is derived from the plasticity chart.

Peat, a highly organic material, is identified as "Pt".

### 4.5 Field Identification of Soils

On preliminary survey, grade construction, and subgrade preparation projects, the materials technologist is required to rapidly classify each soil type encountered.

Section 4.4 outlines the procedures used by the Department's Laboratory to determine the soil classification. In the field it is impractical to run a sieve analysis and to determine the Atterberg Limits of all soil samples.

As a result, a field soils identification test method (ATT-29) was developed. The method uses ten simple hand tests which enables an experienced technologist to rapidly classify a soil with reasonable accuracy. The method provides a means of estimating the Atterberg Limits of the soil, so that the plasticity chart can be used to obtain the classification.

## 4.6 Soil Properties

Soil is a complex construction material. Some of its more important engineering properties are:

1. plasticity,
2. compressibility,
3. permeability,
4. elasticity,
5. cohesion
6. shearing strength
7. shrinkage and swelling
8. frost susceptibility, and
9. erosion resistance.

These properties, when measured and combined, indicate the mechanical and hydraulic characteristics of a soil, and provide the basis for evaluating the soil's suitability for grade construction. Figure 4.7 shows the engineering uses and characteristics of each soil type.

### 4.6.01 Plasticity

Plasticity refers to the ability of a soil to be deformed rapidly without cracking or crumbling, and then maintain that deformed shape after the deforming force has been released. The degree of plasticity exhibited by any soil is related to clay content and moisture content.

### 4.6.02 Compressibility

Compressibility, as a property of a soil, pertains to its susceptibility to decrease in volume when subjected to a loading force.

Compressibility should not be confused with compaction or consolidation. Compaction is the physical act of increasing density by mechanical means by forcing the soil particles closer together thus reducing the air voids. Consolidation is the gradual reduction in the volume of the voids (volume of air+volume of the water), caused by the application of a sustained load and due mainly to the squeezing-out of water from the void spaces.

The compressibility of a soil relates directly to particle size, shape, and orientation of the particles.

Large round particles of equal size, like ball bearings, are essentially incompressible, as the particles can only be arranged in one way. Angular particles of similar size would display more compressibility, as flat surfaces could be arranged so that they lay against each other. The smaller the particles, the greater the number of ways that they can be orientated, consequently, the higher the compressibility. Similarly, the more nonround or the more angular the particles, the greater the compressibility, as more flat surfaces are available.

The plasticity chart shows that compressibility increases with Liquid Limit. Soils composed primarily of silts and clays have a large surface area to hold water, possess high Liquid Limits, and are highly compressible. This is due to the small particle size of silts and clays. Soils composed primarily of gravels and sands, have surface areas which hold little water and therefore possess low Liquid Limits, and are much less compressible due to the larger particle size.

## ENGINEERING USES AND CHARACTERISTICS OF SOILS BASED ON THE UNIFIED SOIL CLASSIFICATION SYSTEM

| Group Symbol Letter | Important Properties When Compacted: Permeability Characteristics k (cm per sec.) | Important Properties When Compacted: Shear Strength (when saturated) | Important Properties When Compacted: Compressibility And Expansion (when saturated) | Workability As A Construction Material | Standard Maximum Dry Density (kg/m³) and Voids Ratio | Compaction Characteristics | Potential Frost Action |
|---|---|---|---|---|---|---|---|
| GW | Pervious $k = 10^{-2}$ | Excellent | Almost none | Excellent | 2 000 - 2 160<br>0.35 | Excellent, tractor, rubber tired, steel wheeled roller | none to very slight |
| GP | Very Pervious $k = 10^{-2}$ | Good | Almost none | Good | 1 840 - 2 000<br>0.45 | Good to excellent, tractor, rubber tired, steel wheeled roller | none to very slight |
| GM | Semi - pervious to pervious $k = 10^{-3}$ to $10^{-6}$ | Good | Very slight to slight | Good | 1 920 - 2 160<br>0.40 | Good to excellent with close control, tractor, rubber tired, sheepsfoot roller | slight to medium |
| GC | Impervious $k = 10^{-6}$ to $10^{-8}$ | Good to Fair | Slight | Good | 1 840 - 2 160<br>0.30 | Excellent, rubber tired, sheepsfoot roller | slight to medium |
| SW | Pervious $k = 10^{-3}$ | Excellent | Almost none | Excellent | 1 760 - 2 080<br>0.40 | Excellent, tractor, rubber tired equipment | none to very slight |
| SP | Pervious $k = 10^{-3}$ | Good | Almost none | Fair | 1 600 - 1 920<br>0.70 | Good to excellent, tractor rubber tired equipment | none to very slight |
| SM | Semi - pervious to impervious $k = 10^{-3}$ to $10^{-6}$ | Good | Very slight to medium | Fair | 1 760 - 2 000<br>0.60 | Good to excellent with close control tractor, rubber tired, sheepsfoot roller | slight to high |
| SC | Impervious $k = 10^{-6}$ to $10^{-8}$ | Good to fair | Slight to medium | Good | 1 680 - 2 000<br>0.35 | Excellent, sheepsfoot roller, rubber tired equipment | slight to high |
| ML | Semi - pervious to impervious $k = 10^{-3}$ to $10^{-6}$ | Fair | Slight to medium | Fair | 1 520 - 1 920<br>0.70 | Good to poor, close control essential, rubber tired roller, sheepsfoot roller | medium to very high |
| CL | Impervious $k = 10^{-6}$ to $10^{-8}$ | Fair | Medium | Good to Fair | 1 520 - 1 920<br>0.70 | Fair to good, sheepsfoot roller, rubber tired roller | medium to high |
| OL | Semi - pervious to impervious $k = 10^{-4}$ to $10^{-6}$ | Poor | Medium to high | Fair | 1 280 - 1 600<br>0.90 | Fair to poor, sheepsfoot roller | medium to high |
| CI | Impervious $k = 10^{-6}$ to $10^{-8}$ | Fair to Poor | Medium to high | Fair | 1 440 - 1 840<br>0.80 | Fair to poor, sheepsfoot roller | medium to high |
| MH | Semi - pervious to impervious $k = 10^{-4}$ to $10^{-6}$ | Fair to Poor | High | Poor | 1 120 - 1 520<br>0.70 | Poor to very poor, sheepsfoot roller | medium to very high |
| CH | Impervious $k = 10^{-6}$ to $10^{-8}$ | Poor | High | Poor | 1 200 - 1 680<br>0.80 | Fair to poor, sheepsfoot roller | medium |
| OH | Impervious $k = 10^{-6}$ to $10^{-8}$ | Poor | High | Poor | 1 040 - 1 600<br>0.70 | Poor to very poor, sheepsfoot roller | medium |
| Pt | Semi - Pervious to impervious, variable | | Very high | | | Compaction not practical | slight |

FIGURE 4.7

### 4.6.03 Permeability

Permeability refers to the ease with which water flows through a soil. Permeability may be decreased by increasing the density of a soil through either:

a) compaction, by forcing the soil particles closer together thereby decreasing the void spaces, or

b) gradation improvement, by filling the void spaces with smaller particles, eg., clays are less permeable than silts; silts less than sands.

### 4.6.04 Elasticity

Elasticity refers to a soil's ability to return to its original shape after being deformed by a load for a short period of time. This property is largely a characteristic of silty soils, and is undesirable in a soil for grade construction.

### 4.6.05 Cohesion and Shear Strength

The shearing strength of a soil is the shear stress (force) necessary to cause slippage on a surface through the soil. The resistance to the stress is caused by friction between the particles, and cohesion. Cohesion is the bond between the particles and is that part of shear strength not due to friction. Cohesion and shear strength are not constant but vary with changes in moisture content, rate and time of loading, confining pressure, and other factors.

### 4.6.06 Shrinkage and Swell

Shrinkage and swell refers to the volume changes which occur in a soil mass without the application or removal of external loads. This is largely a characteristic of highly plastic clays when subjected to changes in moisture content. These materials should be avoided, where possible, in construction.

### 4.6.07 Frost Susceptibility

A frost susceptible soil is one which, when compacted in-place, has void channels within the proper size range to support capillary action. Voids too large (sand) or too small (clay) do not support capillary action. Silt is the most highly frost susceptible soil.

Highly frost susceptible soils react to freezing in an exaggerated and detrimental fashion. This reaction is called frost heaving.

Water expands by about 10% when it freezes. Water freezes in the voids of a soil, expands, and increases the volume of the soil mass. This is not frost heaving.

Frost heaving is caused when water is drawn from the water table to the frost line where it freezes and produces an ice lens as shown in Figure 4.8. The movement of water is caused by capillary rise and continues after the ice lens has formed. As a result the ice lens grows, the soil heaves, and a frost heave is created on the surface. When frost heaving occurs the thickness of the frozen layer of soil increases by 20 to 30%.

**X-SECTION OF A TYPICAL FROST BOIL**

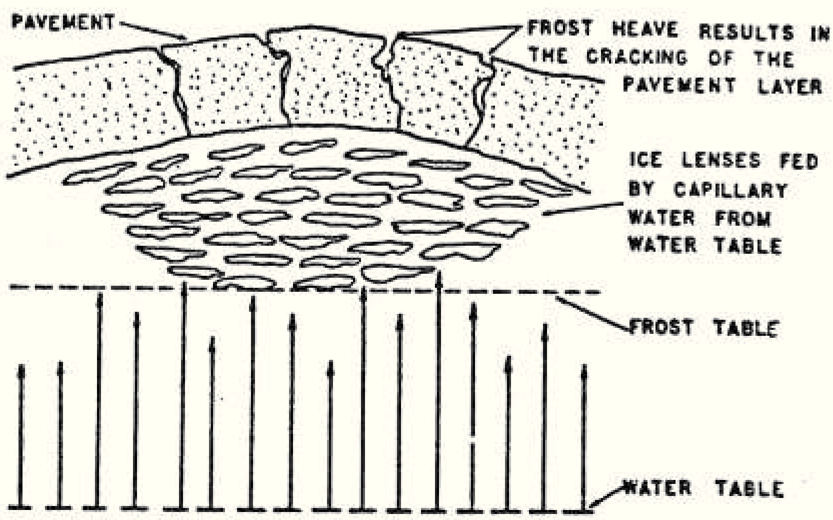



FIGURE 4.8

Three conditions are necessary for frost heaving to occur. They are:

1. Temperature conditions which create a slowly moving frost line,
2. A free water source, and
3. A frost susceptible soil close enough to the water source to be within the range of capillary action.

Due to the movement of water, the moisture content of the soil in the vicinity of the ice lens increases. Subsequent thawing of the soil from the surface downwards will result in a frost boil, as frozen material below will prevent normal drainage. This weakens the surface, often so much that it will not carry traffic.

The potential for frost heaving can be minimized by eliminating one or more of the following conditions:

a) lowering or diverting the water source, or

b) installing an impermeable layer or a layer of clean material incapable of supporting capillary rise between the water source and the susceptible soil, or

c) using soil which is not frost susceptible, or

d) reducing the extent of penetration of the frost line by installing insulation above the frost susceptible soil.

### 4.6.08 Erosion

Erosion is the wearing away of a surface by water currents or flow.

An important consideration in the construction of the highway drainage system is the prevention or control of erosion. Erosion may effect the stability of the pavement by undercutting the pavement from the ditch and shoulder areas. Erosion of cut or fill slopes affects the stability of the embankmentss and backslopes and is a major maintenance problem if not controlled. Eroded materials deposited in a drainage system seriously reduce the system's capacity. The volume of water, the velocity of flow and the soil type are major erosion factors.

Soils containing non-cohesive fine grained particles (i.e., predominantly fine sand and silt soils) are most susceptible to erosion. Particles of coarser materials are heavier and are not so easily transported by water while the cohesive property of clayey soils resist the force of the water flow.

Ditch and shoulder erosion can be controlled by one or a combination of the following:

1. Space ditch inlets at required intervals to check the volume and velocity of the flow.
2. Line or partially line ditches with rip-rap to resist erosion. In some cases rip-rap placed at critical areas, such as culvert and drain inlets and outlets or sharp bends in the ditch alignment, may be most suitable.
3. Line ditches with asphalt or with a half section of corrugated steel pipe.
4. Whenever pavements form a large catchment area on steep grades, intercept the road surface water with a gutter along the pavement edge in order to prevent erosion of the shoulder surface.
5. Flat bottomed ditches are less susceptible to erosion than V-shaped bottoms which also have less drain capacity.
6. Install properly designed weirs (dams) to reduce rate of flow.
7. Use a good grass root system.

## 4.7 Compaction

Compaction is defined as the mechanical process of forcing soil particles closer together. The result is an increase in the soil's density and a decrease in the air voids.

Compaction by increasing density, improves the engineering properties of a soil. The most significant improvements and the resultant effects are summarized below:

| EFFECTS OF PROPER SOIL COMPACTION | |
|---|---|
| IMPROVEMENTS | EFFECT ON ROAD |
| Higher shear strength. | Greater stability. |
| Lower compressibility. | Less settlement under static load. |
| Higher CBR Values. | Less deformation under repeated loads |
| Lower permeability. | Less tendency to absorb water. |
| Lower frost susceptibility. | Less likelihood of frost heave. |

Compaction is controlled on construction projects by comparing the density of the soil placed on the road to a standard density. Since soils vary in particle size and shape and thus compressibility and moisture content, one standard density can not be used for all soils. The technologist determines the moisture-density relationship for each significant soil type by performing test methods ATT-23 or ATT-19 MOISTURE-DENSITY RELATION, Standard Compaction.

### 4.7.01 Moisture-Density Relation Test

ATT-23 and ATT-19 use a standard compactive effort to determine the moisture content at which the maximum density is obtained. In the procedure, a fine grained soil sample is used. In ATT-23, five specimens are formed from the representative material in increments of 2% in moisture content, ranging from below optimum (the moisture content of maximum density) to above optimum. In ATT-19, five specimens are formed in moisture increments of 1.5% ranging from below to above optimum moisture content.

The dry density (the density of the dry soil solids) and moisture content of each of the specimens is calculated, the results are plotted, and the points are joined to form a smooth curve. From the peak of the curve, the density co-ordinate is determined and recorded as the Maximum Dry Density, and the moisture content co-ordinate is determined and recorded as the Optimum Moisture Content. These two results are the compaction standards for that soil type.

### 4.7.01.1 Standard Compaction

The specimens are formed using standard compaction which utilizes a hammer weighing 2.5 kg falling 305 mm. Each specimen is formed in a standard mold in three lifts. Two standard molds are used:

a) The 102 mm diameter mold is used when less than 7% of the soil is retained on the 5000 µm sieve. In this mold, each lift of the specimen receives 25 blows of the standard hammer (ATT-23).

b) The 152 mm in diameter mold is used on materials with more than 70% passing the 20 000 µm sieve. In this mold, each of the three lifts is given 56 blows of the standard hammer (ATT-19).

### 4.7.01.2 Modified Compaction

The modified compaction procedure is not used because it is not part of the Department's General and Construction Specifications. However this condition can occur during construction when compaction equipment compacts the road to a density significantly higher than the standard maximum dry density. This can occur when the soil moisture content is below the optimum moisture content determined from the **Standard** Moisture Density Relation Curve.

Compacted fine grained soils have an equilibrium water content which is above optimum (95-100% saturated). If the soil is at or below optimum, a water deficiency exists and the soil picks up freely available water until it reaches this equilibrium moisture content.

In wet weather, soil in the modified compaction condition would absorb a large amount of water to reach the moisture equilibrium. The soil swells as the excess water pushes the soil particles apart, the soil's original density drops significantly, and the road heaves or breaks up.

This is why modified compaction, i.e., 108% compaction at 4% below optimum, is undesirable. Though the density is above compaction specifications, the moisture content is not. And it is this moisture (or lack of) that will eventually destroy the road.

The modified compaction test procedure utilizes a hammer weighing 4.5 kg falling 457 mm. Each specimen is compacted in 5 lifts at 25 (or 56 for the larger mold) blows per lift. This compactive effort yields a much higher dry density at a lower optimum moisture content.

Figure 4.9 compares the standard to the modified moisture-density relation curve obtained on the same material.

**STANDARD VS MODIFIED MAXIMUM DRY DENSITY**

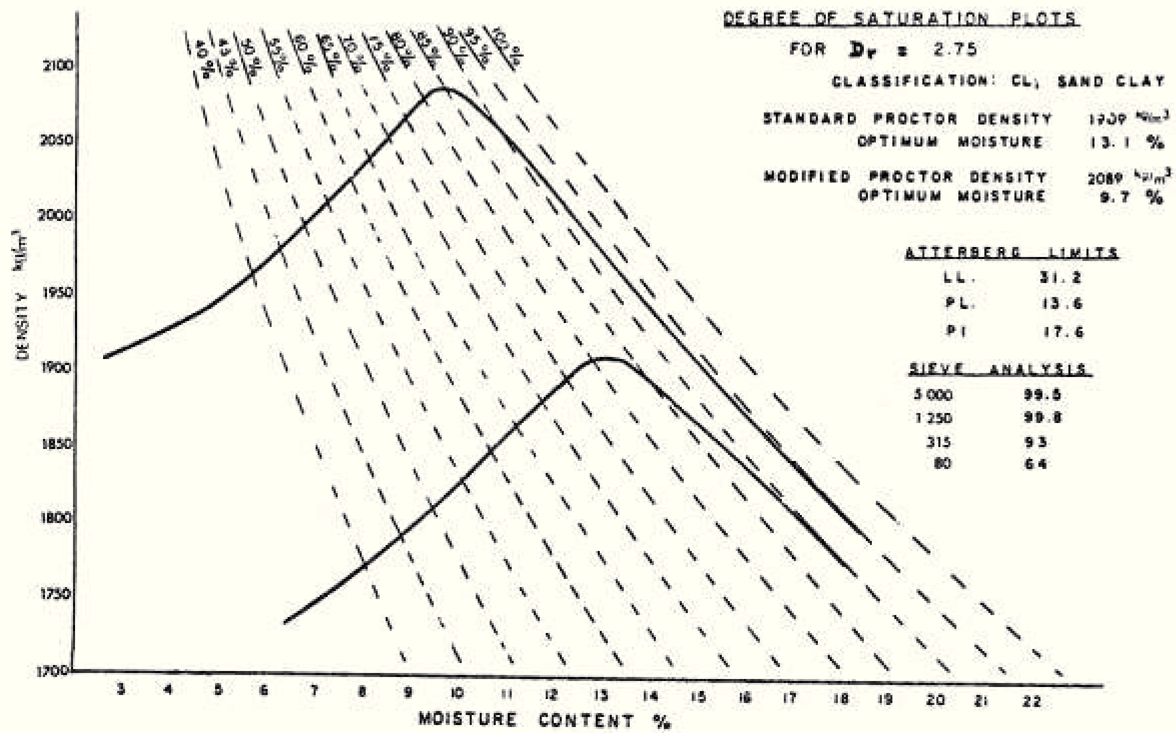



FIGURE 4.9

### 4.7.02 Strength of Compacted Soil

The strength of a compacted soil can be determined by measuring the resistance of that soil to penetration. Soils with high strengths will exhibit high resistance to penetration; soils with low strengths will exhibit little resistance to penetration. This strength can be determined by using the pocket penetrometer.

It is impossible to estimate the degree of compaction or the moisture condition of a compacted soil by penetration resistance methods (pocket penetrometer). Only conventional density tests (sand cone, balloon, nuclear) can be used in the density evaluation of compacted soils. The dry density and moisture content of a soil govern the soil's penetration resistance or strength.

Figure 4.10 shows the effect of varying degrees of compaction effort on the moisture-density relation and penetration resistance for one particular soil. This figure shows that for a given penetration resistance there are an infinite number of combinations of dry density and moisture content.

EFFECT OF COMPACTIVE EFFORT ON THE COMPACTION AND PENETRATION RESISTANCE CURVES

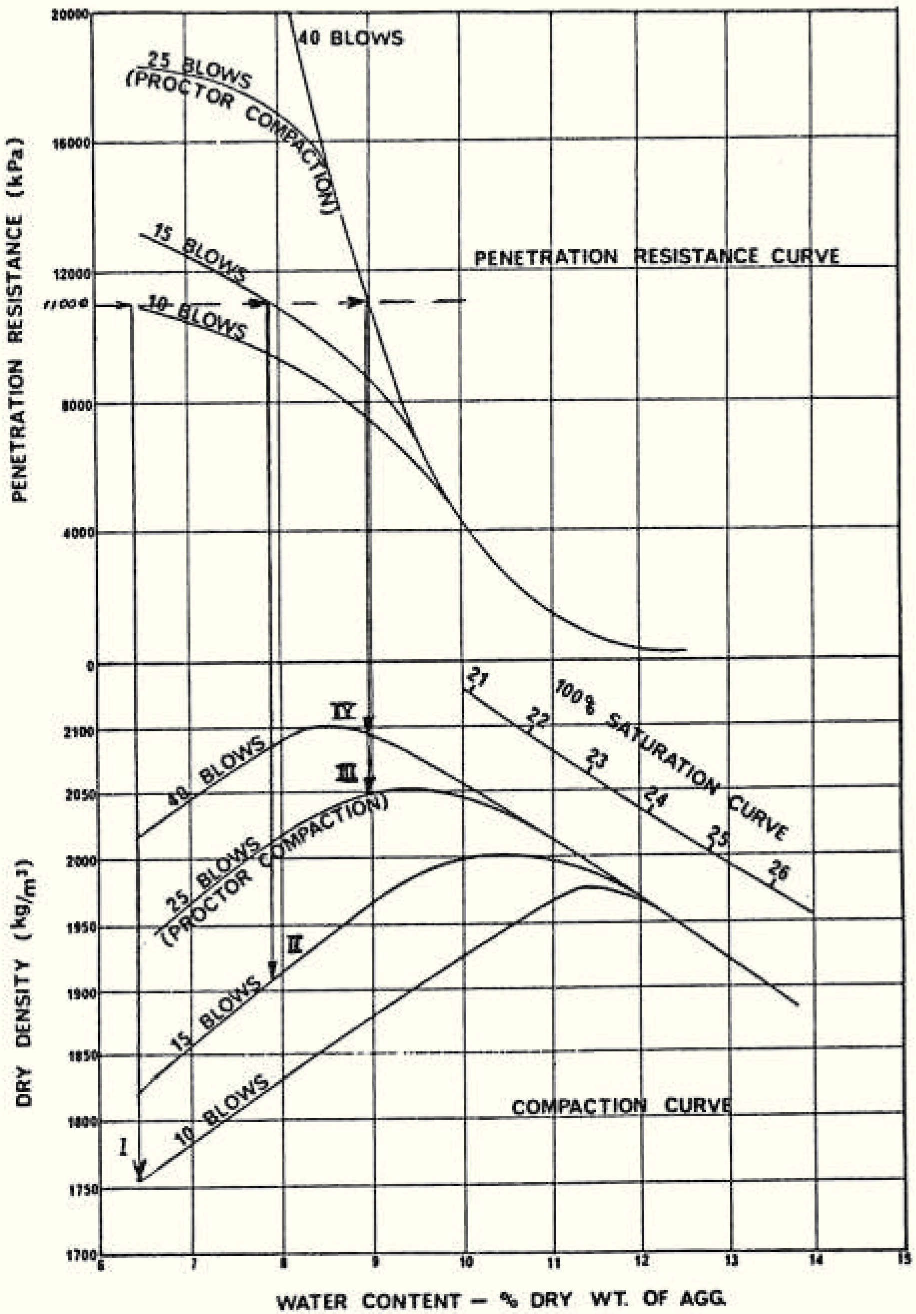



FIGURE 4.10

For example, the following table shows that a penetration resistance of 11 000 kPa could result from a number of moisture-density combinations.

**MOISTURE-DENSITY COMBINATIONS RESULTING FROM A PENETRATION RESISTANCE OF 11 000 kPa**

| Case | I | II | III | IV |
|---|---|---|---|---|
| Penetration Resistance (kPa) | 11 000 | 11 000 | 11 000 | 11 000 |
| Dry Density ($kg/m^3$) | 1 757 | 1 912 | 2 050 | 2 094 |
| Moisture Content (%) | 6.5 | 8.0 | 9.0 | 9.0 |
| % of Standard Maximum Dry Density | 85.6 | 93.2 | 99.9 | 102.0 |
| ± of Standard Optimum Moisture | -2.9 | -1.4 | -0.4 | -0.4 |
| Compactive Effort (blows/lift) | 10 | 15 | 25 | 40 |
| Comparison to Standard Compaction | under | under | standard | over |

In this example, a penetration resistance of 11 000 kPa could result from a soil compacted anywhere between 85.6% of Standard Maximum Dry Density and 2.9% below Standard optimum moisture to 102.0% of Standard Maximum Dry Density and 0.4% below Standard optimum moisture. In other words, there is no direct relationship between soil strength and soil density.

### 4.7.03 Relative Density

In the field, the relative density (abbreviated Dr) of a soil is used to:

a) Plot the zero air voids curve (Section 4.7.04) on the same graph as the moisture density relation curve.

b) Determine the degree of saturation (Section 4.8) of a compacted soil.

c) Determine the volume of the oversize material in the density hole (MEB 2, ATT-8).

The relative density of any substance expresses the weight relationship of a given volume of the substance when compared to an equal volume of water. Water, the universal standard for comparison, has a relative density of 1. Therefore, a material having a relative density of 3, would weigh 3 times more than an equal volume of water. Since 1 $cm^3$ of water weighs 1 g, 1 $cm^3$ of the material would weigh 3 g. As 1 $m^3$ (100 cm x 100 cm x 100 cm) of water weighs 1 000 000 g or 1 000 kg, 1 $m^3$ of the material would weigh 3 times 1 000 kg or 3 000 $kg/m^3$.

Tests conducted on various soil solids show that they possess the following relative densities:

| SOIL TYPE | SYMBOL | RELATIVE DENSITY |
|---|---|---|
| Organic Materials | OL, OH, Pt | 2.45-2.55 |
| Rock | | 2.60 |
| Gravel | GW, GP, GM, GC | 2.65 |
| Sand | SW, SP, SM, SC | 2.65 |
| Silt, low plasticity | ML | 2.65 |
| Silt, high plasticity | MH | 2.70 |
| Clay, low plasticity | CL | 2.70 |
| Clay, intermediate plasticity | CI | 2.75 |
| Clay, high plasticity | CH | 2.75-2.80+ |

### 4.7.04 Zero Air Voids Curve

The zero air voids curve is plotted on the same graph as the moisture-density relation curve.

A compacted soil is essentially composed of three elements: soil solids, water and air (see Figure 4.13). The zero air voids curve represents theoretical densities that would be obtained if a soil could be compacted so that its entire volume was occupied by soil solids and water, in other words, an air voids content of zero.

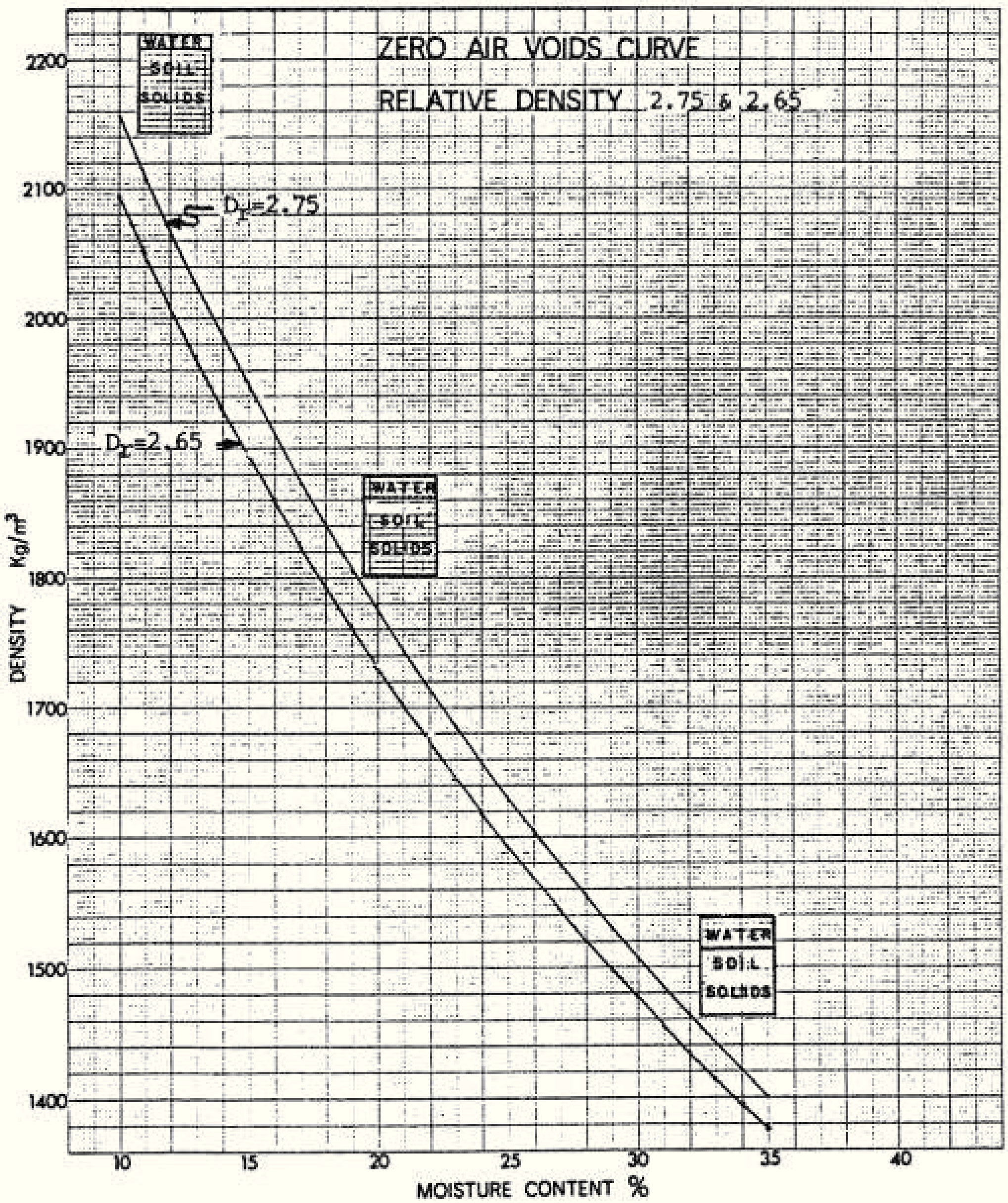



FIGURE 4.11

The curve represents the dry density, therefore, only the weight of the dry soil solids is used to obtain the density. Figure 4.11 shows that at low moisture contents the volume occupied by soil solids is high, while at high moisture contents it is lower.

The density of the dry soil solids at any point on the curve depends on their relative density. The zero air voids curves used in Figure 4.11 are for soils with relative densities of 2.75 and 2.65.

Test method ATT-23 contains zero air voids tables for soils of relative densities of 2.65, 2.70 and 2.75, throughout the common range of moisture contents used in the test. Each table is used for the corresponding soil types.

The zero air voids curve is used to check the moisture-density relation curve. This curve should be parallel to or asymptotic to the zero air voids curve on the wet side of the optimum moisture content. If the moisture-density relation curve touches or crosses the zero air voids curve, an error has been made in the procedure, calculations or plotting.

### 4.7.05 Verification of Moisture-Density Relation Curve Results

Figure 4.12 shows the general shape of moisture-density relation curves for various soil types. This chart can be used as a rough guide in determining if a curve is of the proper shape. Note that:

a) Non-granular highly plastic soils (CH, MH) have relatively flat moisture-density relation curves. These soils have very low maximum dry densities (1500-1600 kg/m$^3$) and very high optimum moisture contents (22-27%).

b) Moisture-density relation curves of granular soils vary with the grain size and the grading. If the granular material is well graded, the curve should have a sharp peak; if the sample is uniformly graded, the curve should have a low flat peak.

Generally the coarser the material, the greater its density and the lower its optimum moisture content.

MOISTURE DENSITY RELATION CURVES FOR VARIOUS SOILS

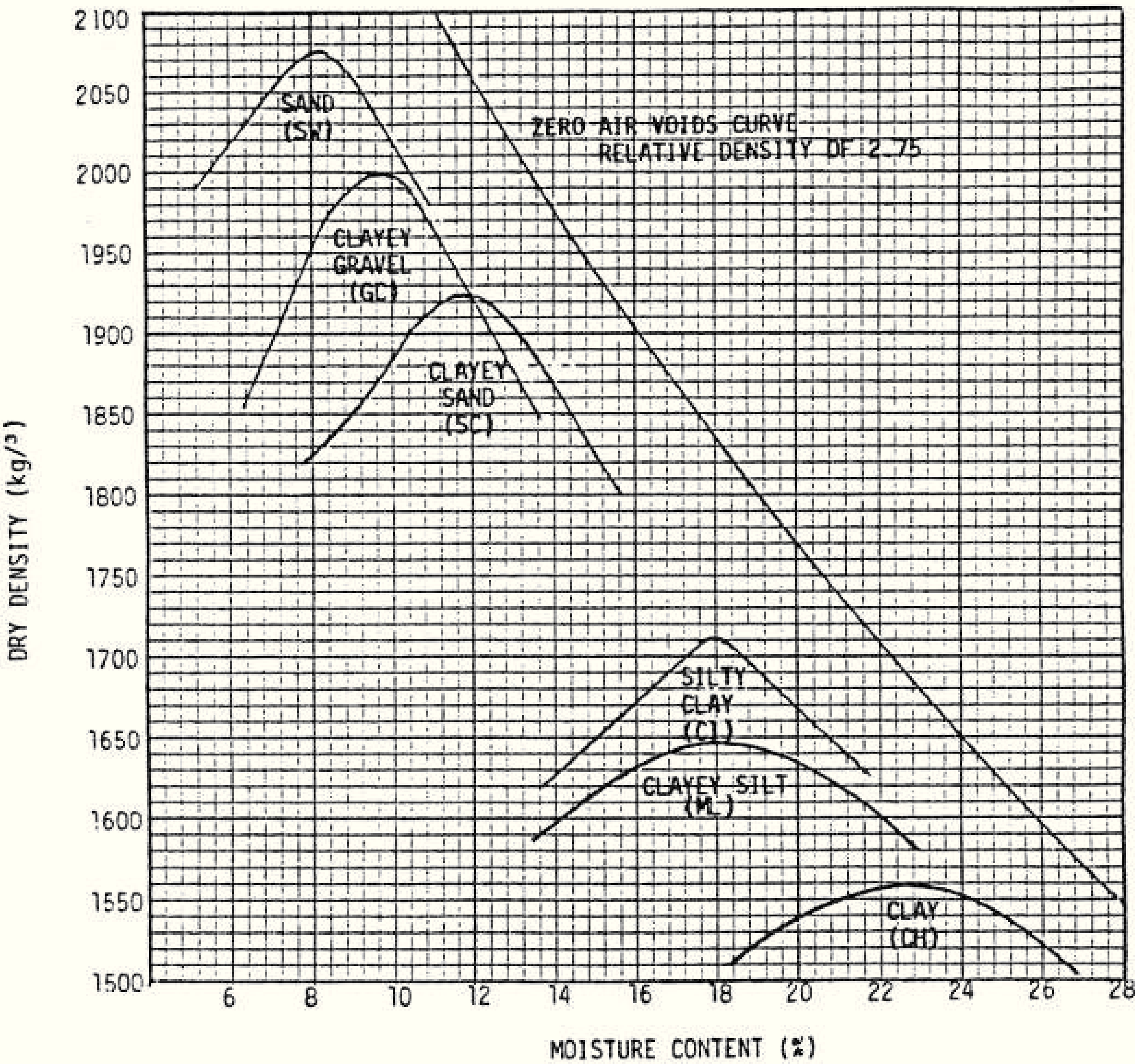



FIGURE 4.12

## 4.8 Degree of Saturation Curves

As a result of compaction, air is expelled from the soil's structure. Therefore, as density increases during compaction, the void space between the soil particles decreases. Since the void space is filled with both water and air, as shown in Figure 4.13, the expulsion of air causes the proportion of the available void space that is filled with water to increase. The degree of saturation (S%) expresses, as a percent, the proportion of the total available void space (Vv) that is occupied by water (Vw): S% - 100 Vw/Vv.

When soil is 100% saturated, all the void space is filled with water, therefore, the air voids content equals zero. Consequently, the 100% degree of saturation curve for a particular soil corresponds to the zero air voids curve for that soil.

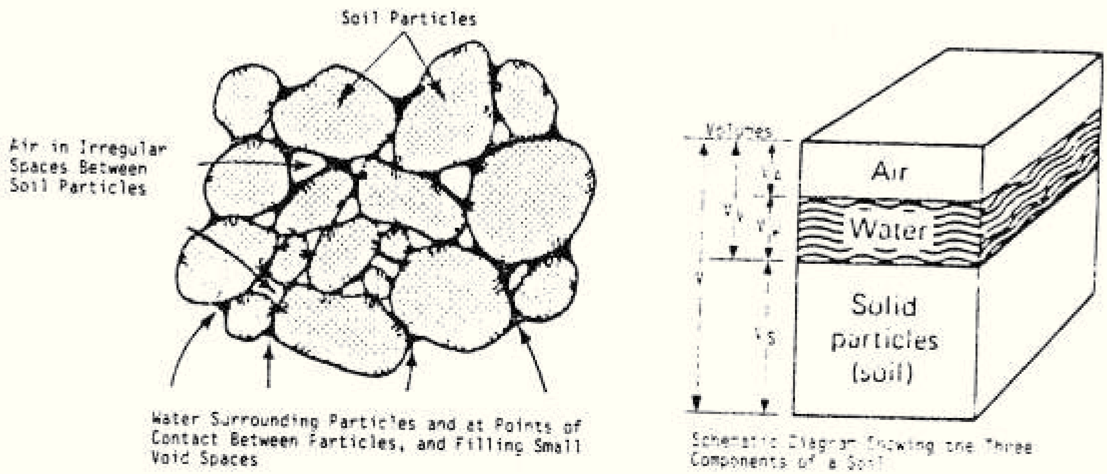



**FIGURE 4.13**

Three sets of degree of saturation curves are supplied to the field, a typical example of which is shown in Figure 4.14. These curves were derived for the normal range of relative densities encountered in materials used in construction. The correct curve to use with a soil depends on the relative density of its soil solids.

If the degree of saturation curves are not available, the degree of saturation (S) in percent can be calculated using the formula:

$$S(\%) = \frac{\text{Moisture Content in \% X Dry Density in g/cm}^3 \text{ X Relative Density}}{\text{Relative Density - Dry Density (g/cm}^3)}$$

i.e., Optimum Moisture Content = 19.4%
Maximum Dry Density = 1663 kg/m³ or 1.663 g/cm³
Relative Density = 2.70

$$S(\%) = \frac{19.4 \times 1.663 \times 2.70}{2.70 - 1.663} = 84\%$$

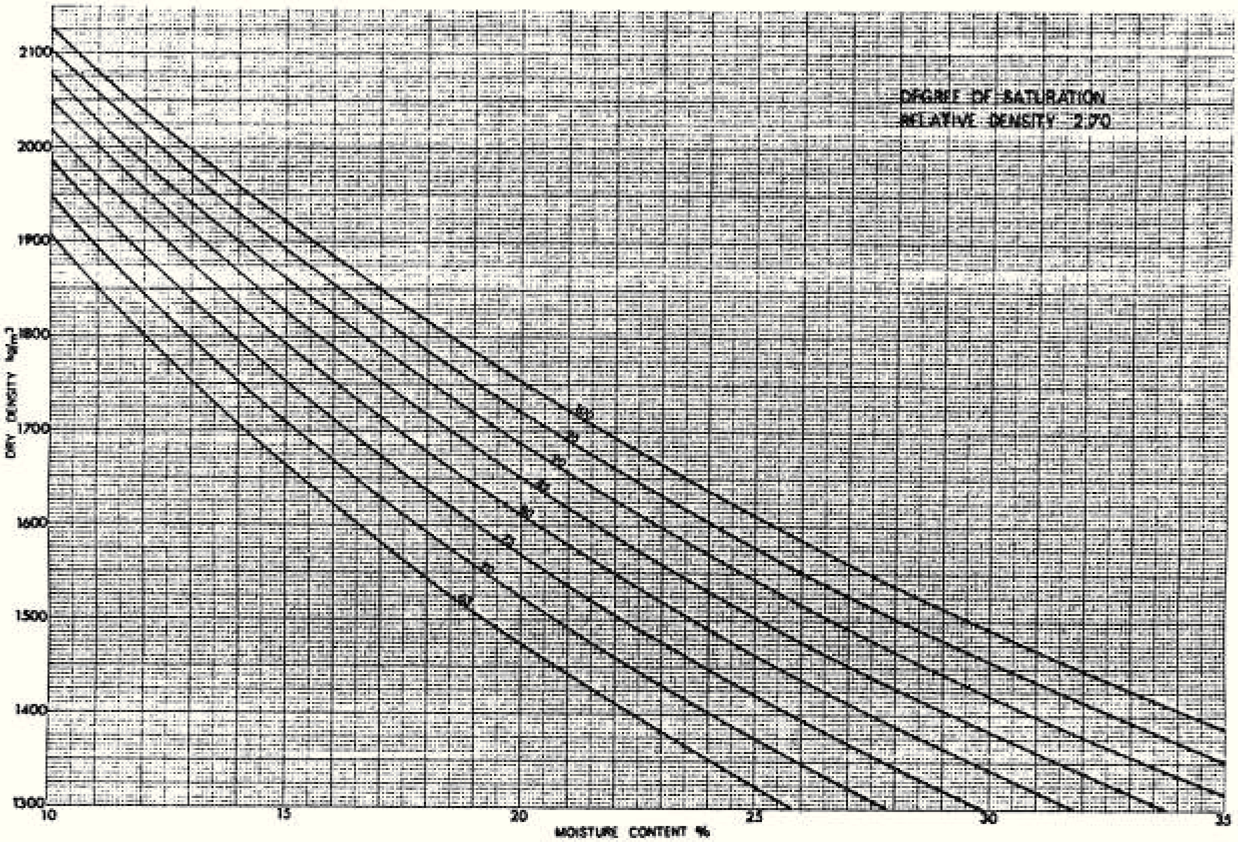



FIGURE 4.14

### 4.8.01 Application

Check the results of each moisture-density relation test curve by determining the degree of saturation of the curve's peak as follows:

a) Plot the maximum dry density versus the optimum moisture content on the appropriate degree of saturation curve, or

b) Calculate the degree of saturation using the curve results and the soil's relative density.

Most soils have a degree of saturation between 80% and 90%, if the soil was compacted at optimum moisture content and standard compaction was used. Sands are closer to 80% while clays are nearer or slightly above 90%. Moisture density relation curves yielding a peak well below or above these limits are in error and should be checked. Points must not cross (or plot near) the 100% degree of saturation curve as it corresponds to the zero air voids curve for a soil.

The proper degree of saturation in the construction roadway ensures moisture stability. If a soil is compacted at near optimum moisture content to the maximum dry density, the degree of saturation is such that the soil is more able to resist changes in moisture content. This condition is reached at a moisture content slightly below optimum for silts and at slightly above optimum for clays. A stable moisture condition eliminates the large volume changes (shrinking and swelling) which can occur with variations in moisture content.

## 4.9 In-Place Density Tests

The Moisture-Density relation of each significant soil should be determined in advance. The technologist should ensure that the moisture content of each soil is close to the optimum moisture content as it is placed into embankments. Once compaction is completed, in-place density tests are performed on the soil as directed in ATT-8, DENSITY, In-Place Balloon Method.

### 4.9.01 Correlation of Field Density to Moisture-Density Relation Test

After the dry density of the in-place soil has been determined, the technologist determines which of the soils used for the moisture-density relation test most closely resembles the density soil. This correlation is the major source of error when the percent compaction is calculated.

Hints which can improve correlation are described in Section 7.6.04.

When the in-place density soil has been correlated to the correct moisture-density relation curve, the percent compaction is calculated.

### 4.9.02 Percent Compaction

Percent Compaction is calculated using the following formula:

$$\text{Percent Compaction (\%)} = \frac{\text{In-Place Dry Density}}{\text{Maximum Dry Density}} \times 100\%$$

# 5.0 PRELIMINARY SURVEY

## 5.1 Purpose

The purpose of the preliminary survey is to acquire geometric and soils survey information along and adjacent to the roadway alignment to:

a) Establish the alignment and elevation of the roadway,
b) Design the drainage along the route,
c) Design the roadway cross-sections, and
d) Estimate the quantity and characteristics of soil materials to be included in the contract.

## 5.2 Soils Survey

Materials technologists on a preliminary survey are required to conduct a soils survey to establish the soil profile throughout the proposed alignment. The field work consists of examining soil obtained by hand or power auger borings, and inspecting excavated areas. The soil changes are identified by performing field hand identification tests. Samples are obtained and submitted to the Transportation Laboratory for analysis and verification of field observations. The purpose of the soil survey is to identify:

a) Depth of topsoil and/or organic matter,
b) Extent and types of subsoil which will be encountered during construction,
c) Location of suitable prospective borrow pits,
d) Subsurface moisture conditions, water table, and drainage requirements,
e) Foundation conditions for embankment and cut slopes,
f) Presence and extent of rock formations,
g) Depth of muskeg areas and nature of underlying soils, and
h) Problems which may have a bearing on the project.

## 5.3 Project Familiarization

Roadway design generally requires input from both district and head office staff. Design Engineering Branch initiates the field work by sending a preliminary survey request form to the District and a copy of the request to the Geotechnical Section. The Geotechnical Section assigns a Geotechnical Engineer to the project. The District and Design Branch are advised as to who is the geotechnical engineer. Open communication between project manager, design engineer and geotechnical engineer helps to identify potential problems and improves the quality of the decisions.

Before starting a soils survey, the project manager, technologist, design engineer and geotechnical engineer should inspect the proposed route and study the existing information on the soil types in the vicinity. For new alignments which parallel the existing road, a survey of soil materials should be made on existing roads. Potential problem areas should be identified and the need for special investigation discussed.

Typical examples of problems are:

- Muskeg/soft soil conditions.
- Springs, seepage zones.
- Gravel seams, coal seams.
- Rock formations.
- Creek or lake crossings, culvert locations.
- High fill areas.
- Deep cut areas.
- Steep side hill fill construction.
- Pipeline locations (especially high pressure type) across the proposed roadway or on the backslopes/side slopes.

Before starting the soil survey, Department personnel must obtain permission from the property owners to enter private property.

Department personnel must contact the proper utility agencies to stake their utility lines to prevent accidents during the preliminary soil survey.

Borings and/or test pits are made at locations that will aid in the designing of the road and be of benefit as a reference during construction.

## 5.4 Soils Log Book

The first page of a level book contains the project number, contract limits, project manager's name and address, and the names of the technologists conducting the survey. The next 4 pages are left blank as they are used later to compile the index.

Each hole is plotted on a separate page and identified by: hole number, station and location. Other necessary information is the date tested, hole depth and weather conditions.

The log book is divided into a borrow pit prospect section and an alignment boring section which includes the boring in rock investigations and muskeg probe data. The recommended format for a typical page is shown in Figure 5.1.

Each hole is plotted on a separate page. The transition depths of the various soil types encountered; field identification results; moisture relationships to optimum; depths at which samples were obtained; sample bag tag numbers; water table level; aggregate; rock formations; boulders; and pertinent comments are recorded. Figure 5.2 shows a completed typical page of the soils log book.

The corresponding Unified Soil Classification Symbol, as shown in Figure 5.3, is used to represent each material.

Upon completion of the survey, the index is compiled on the pages allotted, and the log book turned over to the project manager.

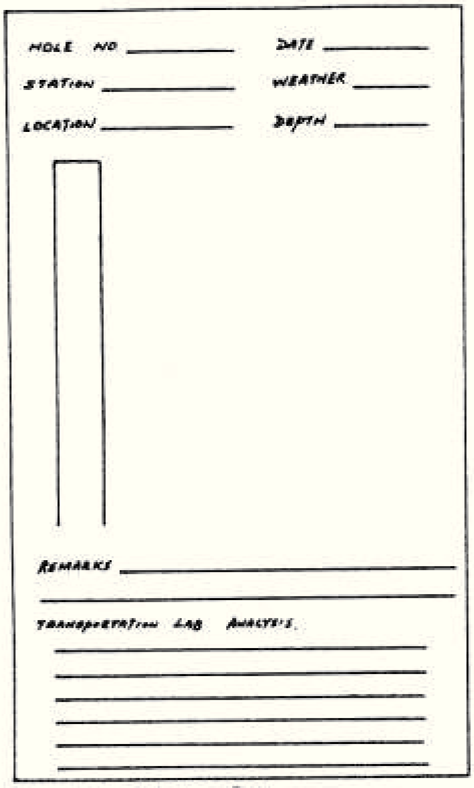



Figure 5.1

TYPICAL SOIL SURVEY HOLE LOG

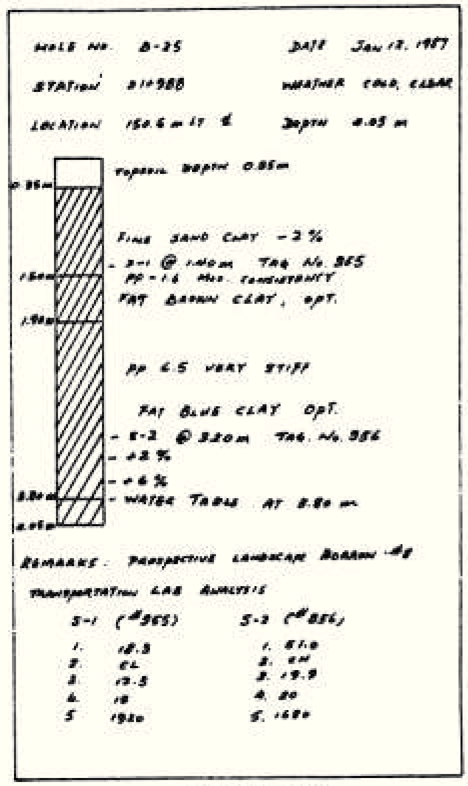



Figure 5.2

UNIFIED SOIL CLASSIFICATION GROUP AND LOG SYMBOLS

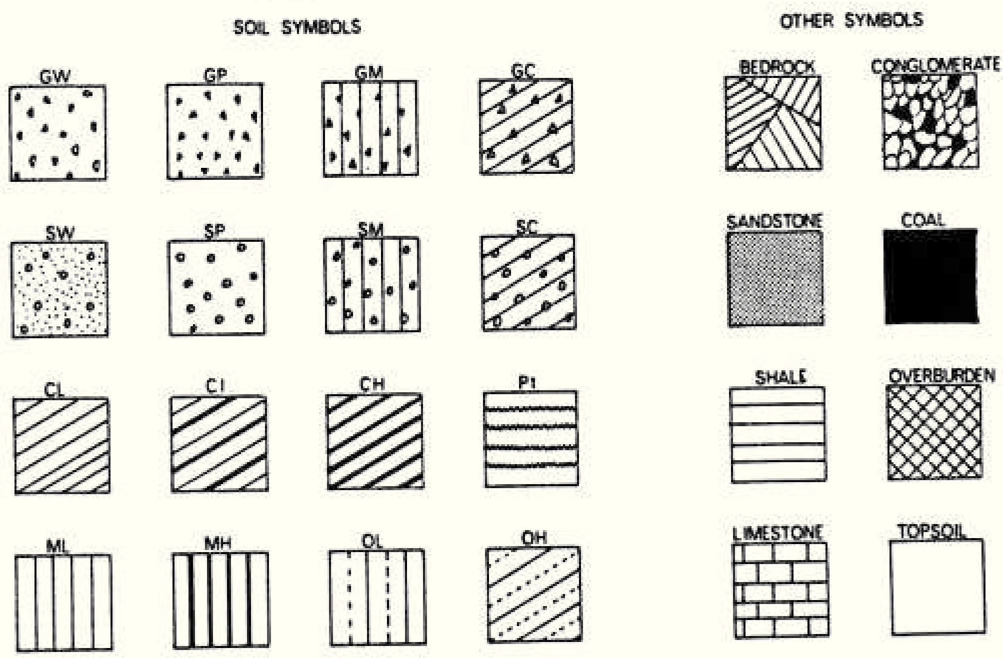



Figure 5.3

## 5.5 Hole Numbers

Each hole boring is given a unique number. Muskeg probes and rock investigations are numbered in the same continuous consecutive series as the alignment borings.

Prospective borrow holes are placed in a separate section of the Soils Log Book and identified by a separate numerical series prefixed with the letter "B", i.e. B1, B2...etc.

## 5.6 Test Borings

### 5.6.01 Location

Excavation areas shall be thoroughly investigated because excavation is the major item used to determine the value of a contract and is normally the material that is used as fill material in subgrade construction. Test sites shall be located laterally within the cross-section over the deepest cut (usually over the design ditch).

When drilling becomes difficult, it is possible either a rock ledge or occasional boulder is present at that depth. It would be advisable to drill more holes in that vicinity to make sure the resistance is fairly uniform at that particular depth. This will indicate a hard layer or a localized boulder or hard dry overconsolidated soil. In order to identify the exact nature of the stiff material, test pits may be needed to complete the preliminary soil survey.

Areas of moderate embankment heights are randomly tested to determine subsurface moisture conditions and thoroughly investigated whenever their stability is doubtful, as explained in Section 5.6.05, DEPTH.

The project soils profile locates the boundaries of significant soil changes. The approximate location of the design shoulder and ditch grade line should be available and the technologist should visualize the finished grade line and determine appropriate test sites. All tested sections are reviewed with the project manager to ensure that sufficient borings are made to the proper depth.

The right-of-way is logged every 200 metres, if the soil is uniform in type, depth, and moisture condition. The terrain and the type of project will determine the offset distance of each test boring. For example, in a grade widening project, it is desirable to drill test borings in the ditch area(s) on one or both sides of the existing road depending on whether widening will be on one side or both sides of the present road. Similarly, for deep cut areas, it is desirable to drill 2 or 3 holes along a cross section at any particular chainage. Do not drill near power, telephone and gaslines.

If the results of adjacent test boring logs differ significantly, then additional locations are chosen to isolate soil type changes. Figure 5.4 is an example of typical test hole locations.

TYPICAL LOCATIONS FOR AUGER BORINGS

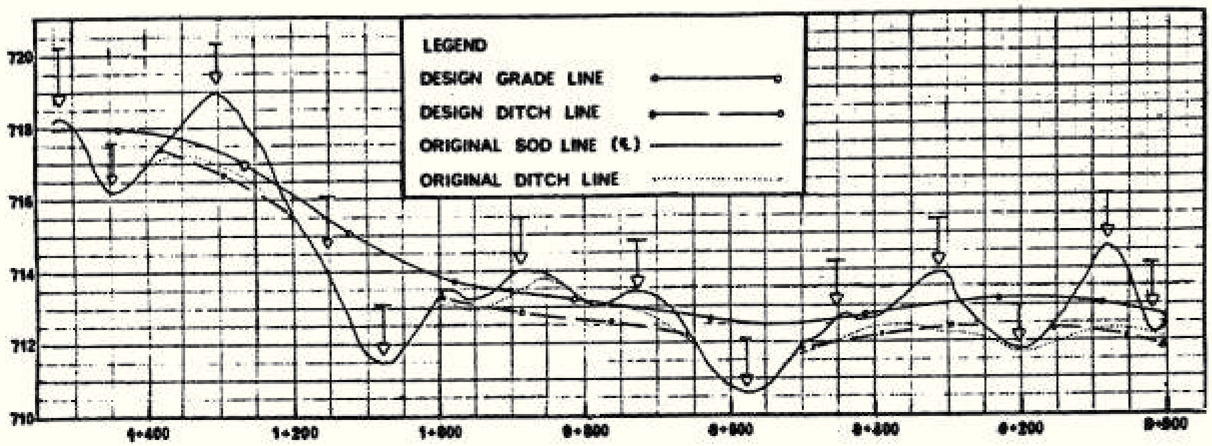



Figure 5.4

Soil changes can influence the stability of high fill areas as well as steep cut sections. Your observations should be noted and the Geotechnical Engineer should be informed of potential stability problems. The information obtained by the pocket penetrometer helps to estimate the stability of a soil and can be used to identify potential problem areas. The Geotechnical Engineer will decide if standard penetration and shelby tube sampling is required.

### 5.6.02 Topsoil

The depth of topsoil is measured to the nearest 0.05 m and recorded in the log book. This information is used to estimate the quantity of waste topsoil in the contract.

### 5.6.03 Field Identification of Soil Samples

For each test hole follow ATT-29, SOILS IDENTIFICATION, Hand Method, to determine the moisture condition and to identify the material at different depths. Use the plastic limit test to relate the soil's observed moisture content to the soil's optimum moisture content. The relationship should be expressed as dry, below optimum, optimum minus, optimum, optimum plus, above optimum, and wet, until proficiency is attained in expressing the relationship as a percentage.

The use of pocket penetrometer is encouraged to determine the natural consistency of clayey soils. As illustrated in Section 5.7.02, the soils can be classified as soft, stiff, hard, etc. These descriptions are not placed on the plan-profile sheets; however, they are useful for the assessment of existing ground conditions when designing the profile grade line. Pocket penetrometer readings should be taken on larger pieces of auger cuttings or material excavated from test pits.