BC HYDRO

WAC BENNETT DAM

EXPERT ENGINEERING PANEL

APPENDICES - VOLUME 2

Kaare Hoeg

Robin Fell

Rodney Bridle

August 2012

Report No. N3405

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APPENDICES in Volume 2

Appendix A: Terms of Reference

WAC Bennett Dam
Expert Engineering Panel on Seepage Flow Control Characteristics

Expectations

The Expert Engineering Panel has been provided with the same raw data as is available to BC Hydro Engineering. So provided, the Panel is to

- independently arrive at an interpretation of the seepage flow control function of the dam's performance

- provide BC Hydro with a basis for determining how well our previous interpretations compare with the Panel's

- determine what further information, analyses and/or performance indicators are required in order to evaluate if or when it would be appropriate to move from a reactive to a proactive approach in regard to remedial work at the dam

The scope of work is different from that of an Advisory Board. Whereas a Board would be asked to opine on mainly on interpretations and courses of action presented to them by BC Hydro, this Panel is to work in detail with the available data, independent from previous BC Hydro interpretations. Later work is expected to include collaborative efforts, such as undertaking potential failure mode analyses.

The overall question for deliberation is "Does the Bennett Dam present any issues that require risk mitigation or investigation at this time, in the context of

- the known distinct defects (such as casings and instrumentation "islands" and trenches, winter horizons etc) and
- general flow control and filtration considerations."

Original: 23 June 2011

Formal and informal questions removed by agreement: 29 February 2012

Appendix B: Information Supplied

Appendix B Information Supplied

1 Supplied on 3 February 2011

WAC Bennett Dam Embankment Dam Project
List of Review Materials for the Expert Engineering Panel

1965-1968 Bennett Dam Construction

- Construction era – Design Basis Report
- Portage Mountain Project Design Report (H1756)
- Papers
  - Ripley C.F. (1967) Portage Mountain Dam I. An Outline of the Project. Canadian Geotechnical Journal, vol. 4, no. 2. 125-138.
  - Morgan G.C., and Harris M.C. (1967) Portage Mountain Dam II. Materials. Canadian Geotechnical Journal, vol. 4, no. 2. 142-166.
  - Low W.I., and Lyell A.P. (1967) Portage Mountain Dam III. Development of Construction Control. Canadian Geotechnical Journal, vol. 4, no. 2. 184-217.
  - Taylor H. (1969) WAC Bennett Dam. Engineering Journal Canada, vol. 52, no. 10, Oct. 1969, 25-34.
- Construction Data
  - Excel Spreadsheets
  - Databooks
  - Logbooks
- Construction Reports

1985 Comprehensive Inspection and Review

- Comprehensive Inspection and Review (CIR) Report

1987 – 1988 WAC Bennett Dam Deficiency Investigations

This project focussed on the unusual pore pressures in the dam

- DI Reports (H1973, H2247)
- Papers
  - Stewart R.A., Imrie A.S., and Hawson H.H. (1990) Unusual Behaviour of the Core at WAC Bennett Dam. Proceedings of the 43 rd Canadian Geotechnical Conference. 549-558.
  - Stewart R.A., and Imrie A.S. (1993) A New Perspective Based on the 25 Year Performance of WAC Bennett Dam. Proceedings of International Workshop on Dam Safety Evaluation, vol. 1. 53-69.

1996 Sinkhole Investigations & Remediation

- Dam Investigation Report (MEP399)
- Geophysics Report (MEP401)
- Dam Characterization Report (MEP400)
- Long Term Monitoring Plan (MEP407)
- Compaction Grouting Project Completion Report (MEP414)
- Blanket Drain Inflow Test Report (MEP452)
- Papers
  - Stewart R.A., and Watts B.D. (2000) The WAC Bennett Dam Sinkhole Incident. Proceedings of the 53 rd Canadian Geotechnical Conference. Specialty Session on the WAC Bennett Dam Sinkhole. Montreal, 39-45.

  - Gaffran P.C., Watts B.D., and McIntyre J.D. (2000) Geophysical Investigation at WAC Bennett Dam. Proceedings of the 53 rd Canadian Geotechnical Conference. Specialty Session on the WAC Bennett Dam Sinkhole. Montreal, 47-55.
  - Sobkowicz J.C., and Holmes A. (2000) Inflow Testing to Determine the Capacity of the WAC Bennett Dam Blanket Drain. Proceedings of the 53 rd Canadian Geotechnical Conference. Specialty Session on the WAC Bennett Dam Sinkhole. Montreal, 57-65.
  - Watts B.D., Gaffran P.C., Stewart R.A., Sobkowicz J.C., and Kupper A.G. (2000) WAC Bennett Dam - Characterization of Sinkhole No.1 . Proceedings of the 53 rd Canadian Geotechnical Conference. Specialty Session on the WAC Bennett Dam Sinkhole. Montreal, 67-75.
  - Garner S.J., Warner J., Jefferies M.G., and Morrison N.A. (2000) A Controlled Approach to Deep Compaction Grouting at WAC Bennett Dam. Proceedings of the 53 rd Canadian Geotechnical Conference. Specialty Session on the WAC Bennett Dam Sinkhole. Montreal, 77-85.
  - Sobkowicz J.C., Byrne P., Leroueil S., and Garner S. (2000) The Effect of Dissolved and Free Air on the Pore Pressures within the Core of the WAC Bennett Dam. Proceedings of the 53rd Canadian Geotechnical Conference. Specialty Session on the WAC Bennett Dam Sinkhole. Montreal, 87-95.
  - Stewart R.A., and Garner S. (2000) Performance and Safety of WAC Bennett Dam A Seven Year Update. Proceedings of the 53rd Canadian Geotechnical Conference. Specialty Session on the WAC Bennett Dam Sinkhole. Montreal, 97-105.
- Advisory Board Reports

1998 – 2001 – Investigations that followed the Sinkhole work (referred to as Project BDPDI) This work focussed on the forensics of the sinkholes – nothing on the rest of the dam. Some of the work carried out herein led to understanding that the problem is likely to be internal erosion and not limited to the benchmark tubes.

- Toe Filter Stage I Construction Completion Report (PSE138)
- Toe Filter Stage II Construction Completion Report (PSE270)
- 2001 Geophysics Report (E001)
- Report on Air Theory (E107)
- Dam Performance Report (MEP402)
- PRT (External reviewers) Reports

2001 Deficiency Investigations (referred to as project GMS01DI)

This project followed Project BDPDI where we looked at the potential global internal erosion problem

- Geophysics Deficiency Investigations Report (E239)
- Report on Monitoring, Testing and Analyses of the Earthfill Dam (E301)
- Report on Interim Assessment of Performance of the Earthfill Dam (E302)
- Canyon Seepage Collection Improvements Construction Report (E468)

Ongoing Work

- Annual Surveillance (including all Monitoring and Surveillance Data)
- Annual Crosshole Reports (2001 to 2010)
- Independent Reviews of Surveillance Reports (DiBiaggio 2001 to 2006 and Rollie Peggs 2009)

2 Papers and Reports supplied on 11 March 2011

SENT TO EXPERTS	WAC BENNETT DAM - PAPERS	
		authors
15-Feb	Advances in Crosshole Seismic Intrumentation for Dam Safety Monitoring	Anderlini-Taylor
11-Mar	Installation of Multi Level Piezometers in an Existing Embankment Dam	Baker
11-Mar	Surveillance-The Cornerstone of Dam Risk Management	Stewart-Garner-Scott-Baker
11-Mar	On the internal Stability of Granular Soils	Fannin-Moffat
11-Mar	The Decommissioning of Coursier Dam – A case for Dam Safety	Garner-Seyers-Matthews
15-Feb	Advances in Crosshole Seismic Instrumentation for Dam Safety Monitoring	Anderlini-Taylor
15-Feb	Understanding internal erosion-A decade of research following a sinkhole even	Garner-Fannin
11-Mar	Crosshole Seismic Measurements to Characterise…Embankment Dams	Vazinkhoo-Gaffran
15-Feb	Advances in Crosshole seismic measurements	Vazinkhoo-Anderlini-Jefferies-Gaffran
15-Feb	Internal Instability in Gap-Graded Cores and Filters	Garner-Sobkowicz
11-Mar	Numerical modeling of suffusion as an Interfacial erosion process	Golay-Bonnelli
11-Mar	Spatial and temporal progression of internal erosion in cohesionless soil	Moffat-Fannin-Garner_
11-Mar	Numerical model of suffusion in terms of finite element method	Popielski-Dluzewski-Stasierski

SENT TO EXPERTS	WAC BENNETT DAM - PAPERS	
		authors
11-Mar	Surveillance Improvements at a Large Canadian Dam	Scott-Hill
11-Mar	Prediction and Validation of Compaction Grout Effectiveness	Shuttle-Jefferies
11-Mar	Anomalous Pore Pressures in earth Dams	Sobkowicz-Garner
11-Mar	Forces and Confining Pressure Effects on Piping Erosion	Stewart S
11-Mar	WAC Bennett Dam - The Sinkhole Crisis	Stewart
15-Feb	A new perspective Based on the 25 year performance	Stewart-Imrie
15-Feb	Unusual Behaviour of the Core at WAC Bennett Dam	Stewart-Imrie-Hawson
11-Mar	Bennett Dam Sinkhole Investigation	Stewart-Watts
15-Feb	WAC Bennett Dam	Taylor
11-Mar	Compaction Grouting for Sinkhole Repair at WAC Bennett Dam	Warner-Jefferies-Garner
11-Mar	Modeling of thermomechanical behaviour of embankment dams	Wilmanski
15-Feb	Geophysical Investigations at WAC Bennett Dam	Gaffran-Watts
15-Feb	Portage Mountain Dam_1_An Outline of the Project	Ripley
15-Feb	Portage Mountain Dam_2_Materials	Morgan-Harris

SENT TO EXPERTS	WAC BENNETT DAM - PAPERS	
		authors
15-Feb	Portage Mountain Dam_3_Development of construction Control	Low-Lyell
15-Feb	Design, monitoring and maintaining drainage system	Taylor-Chow
15-Feb	Measures taken to limit the possible development of cracks	Taylor-Morgan
11-Mar	The laws of proportioning concrete.pdf	Fuller-Thompson
11-Mar	Johansson_Evaluation of Temperature measurements and Seepage monitoring systems	Johansson
11-Mar	The Peace River Project- From feasibility Report to First Power Output	Miles
11-Mar	Kenney-Lau_Internal Stability of Granular Filter	Kenney-Lau
11-Mar	WAC Bennett Dam - Review of WAC Bennett Dam Internal Instability research project	MuirWood-Charles
11-Mar	Examination of Foundation and Dam core material	
11-Mar	Experience in Automated Dam Monitoring Case History - WAC Bennett Dam (Canada)	
11-Mar	MPS article - Repairs start at Bennett Dam	

SENT TO EXPERTS	WAC BENNETT DAM - REPORTS	DIGITAL copy
15-Feb	Report E239_Geophysics Deficiency Investigations_2004	✓
11-Mar	Report E301_Report on Monitoring, Testing and Analyses of the Earthfill Dam_Vol 1 and 2	✓
11-Mar	Report E302_Report on Interim Assessement of the Performance of the Earthgill Dam	✓
15-Feb	Report E468_Canyon Seepage Collection Improvements- Construction report	✓
11-Mar	Report E610_Mica Dam Deficiency Investigation_Crosshole Seismic Testing - Summary report_2008	✓
11-Mar	Report E647_Mica Dam Deficiency Investigation_2008 Crosshole Seismic Testing - Summary report	✓
11-Mar	Report E865_WAC Bennett Dam _Enhanced Surveillance and Response Plan Sinkhole No2 Data review_2010	✓
11-Mar	Report H735_Lower Peace River_Memorandum on Gravel Deposit_1974	✓
15-Feb	Report H1756_Portage mountain Project_Design Report_1988	✓
15-Feb	Report MEP399_WAC Bennett Dam - Sinkhole Investigation Report_2000	✓
15-Feb	Report MEP400_WAC Bennett Dam - Sinkhole Characterization Report_1999	✓
15-Feb	Report MEP401_WAC Bennett Dam_Sinkhole Inestigation_Geophysics Report_1999_Vol 1 and 2	✓
11-Mar	Report MEP402__WAC Bennett Dam_Dam performance report_2001	✓
11-Mar	Report MEP407 - Long Term Monitoring Plan_2000	✓

SENT TO EXPERTS	WAC BENNETT DAM - REPORTS	DIGITAL copy
15-Feb	Peggs_WAC Bennett Dam Surveillance Program – 2009 External Review	✓
15-Feb	Report MEP414_WAC Bennett Dam - Sinkhole Remediation Project - Compaction Grouting report_1998	✓
15-Feb	Report MEP452_WAC Bennett Dam - Sinkhole Investigation - Blanket Drain Inflow Test Report_1999	✓
11-Mar	Report MER-2007-024_WAC Bennett Dam - 2006 Horizontal and Vertical Surveys	✓
11-Mar	Report MER-2007-085_WAC Bennett Dam - 2007 Horizontal and Vertical Surveys	✓
11-Mar	Report MER-2008-072_WAC Bennett Dam - 2008 Horizontal and Vertical Surveys	✓
11-Mar	Report MER-2010-012_WAC Bennett Dam - 2009 Horizontal and Vertical Surveys	✓
11-Mar	Report N1721_WAC Bennett Dam- Crosshole Shear-Wave Tomography- Final report	✓
15-Feb	Report PSE138_Toe Filter Stage 1 - Construction report	✓
15-Feb	Report PSE270_Toe Filter Stage 2 - Construction report	✓
11-Mar	Report TDB_1006_WAC Bennett Dam_Technical Data Book_1985_Vol 1 and 2	✓
11-Mar	Report_20236-11_Portage Mountain Dam - Preliminary report on basic design data_1963	✓
11-Mar	Report_20236-12_Portage Mountain Development- Design report on powerhouse and associated structures_1964	✓
11-Mar	Report_20236-13_Portage Mountain Dam- Notes on the design of the underground powerhouse_1964	✓
11-Mar	Report_H155_Design Report on the Core Contact excavation for the left abutment_1963	✓

SENT TO EXPERTS	WAC BENNETT DAM - REPORTS	DIGITAL copy
15-Feb	Report_H1752_Comprehensive Inspection and Review_1984	✓
11-Mar	Report_H1973_Dam Safety Investigations_WAC Bennet Dam - Report on 1987 Investigations_1987_Vol 1 to 4	✓
11-Mar	Report_H2247_Dam Safety Investigations_WAC Bennet Dam - Deficiency Investigations_1990_Vol 1 and 2	✓
11-Mar	Report_H2491The Detail Design of the Grout Blanket in the Dam-Core-Contact Area_1964_V1	✓
11-Mar	Report_H73_Portage Mountain Dam - Interim Design Report_1962	✓
11-Mar	Report H185 Embankment and Foundation Instrumentation - Interim Design Report_1964	✓
11-Mar	Report H182_Portage Mountain Dam - Fill placement quality control_1964	✓
11-Mar	Report H583__Portage Mountain Development_Instrumentation Observations_Supplemental report No5_1971	✓
11-Mar	Report H684_Portage Dam Development_Instrumentation Observations_1973	✓
15-Feb	Report E001_Dam Safety_Deficiency Investigation_2001 Annual Geophysics Report	✓
15-Feb	Report E107_Dam Safety_Report on Air Theory and Fines Migration Testing	✓
15-Feb	Report E117_Annual Crosshole Seismic Survey_2002	✓
15-Feb	Report E224_Annual Crosshole Seismic survey 2003	✓
15-Feb	Report E316_Annual Crosshole Seismic survey 2004	✓
15-Feb	Report E409_Annual Crosshole Seismic survey 2005	✓

SENT TO EXPERTS	WAC BENNETT DAM - REPORTS	DIGITAL copy
15-Feb	Report E515_Annual Crosshole Seismic survey 2006	✓
15-Feb	Report E615_Annual Crosshole Seismic survey 2007	✓
15-Feb	Report E681_Annual Crosshole Seismic survey 2008	✓
15-Feb	Report E794_Annual Crosshole Seismic survey 2009	✓
15-Feb	Site Instrumentation and Inspection Manual - August 1998 Vol 1 to 3	✓
11-Mar	GMS96DIS-D209.3 Vol1_WAC Bennett Dam - Donald Bruce - Specialist Consultant Reports	✓
11-Mar	GMS96DIS-D209.5 Vol1_WAC Bennett Dam - Cam Kenney - Specialist Consultant Reports	✓
11-Mar	GMS96DIS-D113.4_Permeability Characteristics for Zones 1,2,3 and 6 Material of the Portage Mountain Dam	✓
11-Mar	GMS96DIS-D113.4_Memorandum on Triaxial Testing of Zones 1 and 6 Materials 1965	✓
11-Mar	GMS96DIS-D113.4_Zone 6 Sand Consolidated Drained Triaxial Test 6-D and 6-D2	✓
11-Mar	GMS96DIS-D113.4_IPEC_Factors Affecting The Hydraulic Stability of Filter (zone3)	✓
11-Mar	GMS96DIS-D113.4_Permeability Tests on South Moraine Materials_1963	✓
11-Mar	Technical Notes on Peace River Project_1965	✓
15-Feb	Surveillance program 1997 - Annual Report	✓
	Surveillance program 1998 - Annual Report	HARD COPY ONLY

SENT TO EXPERTS	WAC BENNETT DAM - REPORTS	DIGITAL copy
15-Feb	PSE285_Surveillance program 1999 - Annual Report	✓
15-Feb	PSE351_Surveillance program 2000 - Annual Report	✓
15-Feb	PSE416_Surveillance program 2001 - Annual Report	✓
15-Feb	N2128_Surveillance program 2002 - Annual Report	✓
15-Feb	N2388_Surveillance program 2003 - Annual Report	✓
15-Feb	GEN28_Surveillance program 2004 - Annual Report	✓
15-Feb	GEN39_Surveillance program 2005 - Annual Report	✓
15-Feb	1996_Sinkhole Investigation_Advisory Board Report_1 to 9	✓
11-Mar	PRT Report	✓
15-Feb	Annual Surveillance review 2001-2006_Emilio di Biaggio	✓
11-Mar	Report_Grout Blanket - Construction report, technical_1964	✓
11-Mar	Report_WAC Bennett Dam_Examination of foundation and dam core materials_1987	✓
11-Mar	Report_Portage Dam Development - Report on Instrumentation readings_1968	✓
11-Mar	Report_Portage Dam Development - Report on Instrumentation readings_1968_supl No2	✓
11-Mar	Report_W.A.C. Bennett Dam - basic designs	✓

SENT TO EXPERTS	WAC BENNETT DAM - REPORTS	DIGITAL copy
11-Mar	Report_Frost Penetration in Portage Mountain dam- Winter 1964-1965	✓
11-Mar	Report on Critical State Testing Results of WAC Bennett Dam Core	✓
11-Mar	D244.11_Proposed revision to take limits during compaction grouting_Inter-office memo - Mike Jefferies	1
11-Mar	C112_Data summary of pressumeter testing at Canoe Pass Terminal_Roberts Bank	1
11-Mar	C115_Data summary of Pressuremeter Testing at the West Sinkhole at WAC Bennett Dam	1
11-Mar	Embankment Fill Progress Report_16-31 october 1964	1
11-Mar	Construction Progress Report No1_1965	1
11-Mar	Construction Progress Report No2_1965	1
11-Mar	Construction Progress Report No3_1965	1
11-Mar	Construction Progress Report No4_1965	1
11-Mar	Effect of the Fines Content Loss on Collapse Potential	1
11-Mar	Elastic Properties of Bennett Dam Core and Transition	1
11-Mar	Mamorandum on Analysis of Fill Quality Control for 1966	1
11-Mar	Fill Placement Quality Control_Analysis of Field Tests for 1965	1
11-Mar	Report on Portage Mountain Dam Zone 1_High pressure Triaxial Tests	1

SENT TO EXPERTS	WAC BENNETT DAM - REPORTS	DIGITAL copy
11-Mar	Fill Placement Quality Control_Memorandum on general Principles and Methods	1
11-Mar	Report on Portage Mountain Dam Zone 6 Sand consolidated drained Triaxial Tests	1
11-Mar	Permeability Characteristics For zones 1-2-3-6 Material_1965	1
11-Mar	Report of Triaxial Compression Test 4-1_zone4_1965	1
11-Mar	Report of Triaxial Compression Test_zone1_1965	1
	Report MEP504_WAC Bennett Dam - Reservoir Slopes_Annual Surveillance report_1997	HARD COPY ONLY
	Report PSE214_WAC Bennett Dam - Reservoir Slopes_Annual Surveillance report_1998	HARD COPY ONLY
	D110.4_WAC Bennett Dam - Sinkhole Investigation - Advisory Board Meeting No7	HARD COPY ONLY
	D110.2_WAC Bennett Dam - Sinkhole Investigation - Advisory Board Meeting No5	HARD COPY ONLY
	D110.2_WAC Bennett Dam - Sinkhole Investigation - Advisory Board Meeting No4	HARD COPY ONLY
	D113.1_WAC Bennett Dam_ Sinkhole Investigations_Reports, Specs and Correspondence	HARD COPY ONLY
	C-115_Data Summary of Pressumeter Testing at the West Sinkhole at WAC Bennett Dam_1997	HARD COPY ONLY
	Report N2102_Two Dimensional Stress Analysis of Bennett Dam for Sinkhole Investigation_1997	HARD COPY ONLY
	Report N2069_Shear Wave Seismic Tomography Investigations at the WAC Bennett Dam_1997	HARD COPY ONLY
	Report N2096_WAC Bennett Dam - Sinkhole Investigation - Laboratory Wetting/Collapse tests_1996	HARD COPY ONLY

SENT TO EXPERTS	WAC BENNETT DAM - REPORTS	DIGITAL copy
	Report N2084_Ground Penetration Radar - Investigation at the WAC Bennett Dam Vol1 of 2	HARD COPY ONLY
	Report N2100_Instrumentation Response to Drilling Activities_D245.3.3_DRAFT_1997	HARD COPY ONLY
	Report N2076 WAC Bennett Dam Investigation - Presentation of Seismic CPT Test results_1996	HARD COPY ONLY
	Report OMSGMS_Operation, Maintenance and Surveillance Manual for Dam Safety	HARD COPY ONLY
	Report H2489_Portage Mountain Dam - Curtain Grouting - Technical Construction report_1967	HARD COPY ONLY
	Report MEP250 Summary of information for Technical Review group Meeting No1_1996	HARD COPY ONLY
	Report N2077 Geophysical surveys - Sinkhole Investigation - Figues_1996 (2 Vol)	HARD COPY ONLY

Appendix C: Development, Design and Construction of WAC Bennett Dam 1964-69

APPENDIX C

DEVELOPMENT, DESIGN AND CONSTRUCTION OF WAC BENNETT DAM 1964-69

CONTENTS

APPENDIX C

DEVELOPMENT, DESIGN AND CONSTRUCTION OF WAC BENNETT DAM 1964-69

C1 INTRODUCTION

This appendix is intended to be a record of the intentions, knowledge and standards of the engineers, geologists and contractors responsible for the design and construction of the WAC Bennett Dam. The information used comes mainly from papers written by them during and soon after construction. As the work of several authors is reported, there is some repetition. Notes have been made in the text where it has become apparent that construction details differ from those described in the papers. Many aspects, such as embankment and foundation permeability, for example, have been re-examined and reported on by the EEP.

C2 GENERAL DESCRIPTION

WAC Bennett Dam, formerly known as Portage Mountain Dam, is an essential component of the Peace River Project, described by Ripley (1967)[1] and in more detail in Report H1756 (1988)[2]. The dam is a 600-ft high earthfill structure, 6,700-ft long. The dam crosses the Peace River Canyon and the terraces to the right and left of it. The length of the dam across the canyon is about 1,000-ft on the dam axis, about 3,500-ft on the right terrace which has a maximum height at the canyon edge of about 400-ft, and about 2,000-ft on the left terrace with a maximum height at the canyon edge of about 200-ft. The intake penstocks pass under the right terrace and it houses the underground 2,270 MW installed capacity power station. The crest level varies from 2230-ft at the abutments to 2234-ft above the canyon. The fill volume is 57.5 million yds^3. The reservoir with top water level elevation of 2,200-ft is 225 miles long and provides 41 x 10^9 m^3 (35.5 million acre-feet) of active storage. Maximum flood level was 2215-ft, normal maximum is 2205-ft. Recently, maximum water level has been restricted to 2205-ft by operating the spillway gates if the level is expected to rise above 2205-ft.

Filling commenced in November 1967 and reached full storage capacity (Elevation 2,205-ft, 672 m) in the summer of 1970[3].

As shown on the figure below, the dam is sited in the Peace River Canyon, which is at the end of broad east-west valley passing through the generally northeast-south west Rocky Mountain ridges. Most of the reservoir is situated in the Rocky Mountain Trench upstream of the valley through the ridge. Downstream of the dam the valley widens and the Peace River (on which there are existing hydropower dams with more planned) flows to the east passing by Hudson's Hope (about 12 miles downstream of the dam), Fort St John (about 30 miles downstream of the dam), Fairview, Grimshaw and Peace River (about 175 miles downstream of the dam), where it turns northwards and then east near Fort Vermilion and discharges into Lake Athabasca near Fort Chipewyan about 520 miles downstream of the dam. Lake Athabasca discharges along the Slave River to the Great Slave Lake and from there to the Arctic Ocean.

[1] Ripley C F (1967) Portage Mountain Dam I. An Outline of the Project. Canadian Geotechnical Journal, vol IV, no 2 (on CD1 March 2011).
[2] Report H1756 (1988) Portage Mountain Project Design Report (on CD1 March 2011)
[3] Taylor H & Chow Y M (1976) Design, monitoring and maintaining drainage system of a high earthfill dam. ICOLD 12th Congress Mexico Q45 R10 pp147-167.

C3 SITE GEOLOGY

At the dam site the bedrock consists of interbedded sandstones, shales and occasional coal seams of Lower Cretaceous age. The strata are relatively flat lying and except for occasional mylonite seams are completely free of faults. The dip is 5-10° downstream, and the strike is northwest-southeast and makes an angle of about 40° with the dam axis. Consequently the upper layers 'daylight' into the dam foundation and into the bed of the reservoir.

The Dunlevy formation sandstones are thick-bedded, with few joints and no tendency to weather on exposure. They are predominant to a height of about 200-ft above the canyon floor. The Gething formation shale beds predominate at higher levels. The shales are thin-bedded and have a more highly developed pattern of jointing than the sandstones. They tend to weather and disintegrate into small flakes and granules on exposed surfaces. Coal seams, generally 1-in to 12-in thick, are common within the shale bands, more frequent above about mid-height of the dam. A major coal bed, the Peace River Coal, 7-10-ft thick occurs at about mid-height of the dam. It was mined in a mine about 1000-ft by 400-ft in area beneath the downstream toe of the left abutment of the dam, which was backfilled tightly with gravel, and the mine entrances sealed with concrete during construction.

C4 MYLONITE SEAMS

The mechanics of the formation of what 'the geologists called mylonite seams' were not fully known (in 1967). They were believed to be planes of weakness along which small shearing strains from regional warping had occurred to relieve stresses created in the bedrock by regional warping. The Dictionary of Geology (1996)[4] confirms that that this was indeed what had created them as it describes mylonite as 'fine-grained foliated rock with recrystallised texture with 50-90% matrix and a strong lineation caused by shear in a major ductile fault or shear zone'. The Design Report notes that the mylonites here are not recrystallised and therefore not true mylonites. It describes the 'mylonites' as 'bedding plane seams of gouge and breccia'. There is an extensive mylonite seam in the right bank foundation. It is exposed in the tunnels below the dam, and may have an influence on overall stability because friction angles in mylonite, which is sometimes reduced (as at Bennett) to 'pulverized fine powder', are commonly at residual. The Design Report gives ø' of 11-31° with an average of 21° in the shale mylonite, and 28-30° in the coal mylonite.

[4] The New Penguin Dictionary of Geology (1996) Penguin Books, London

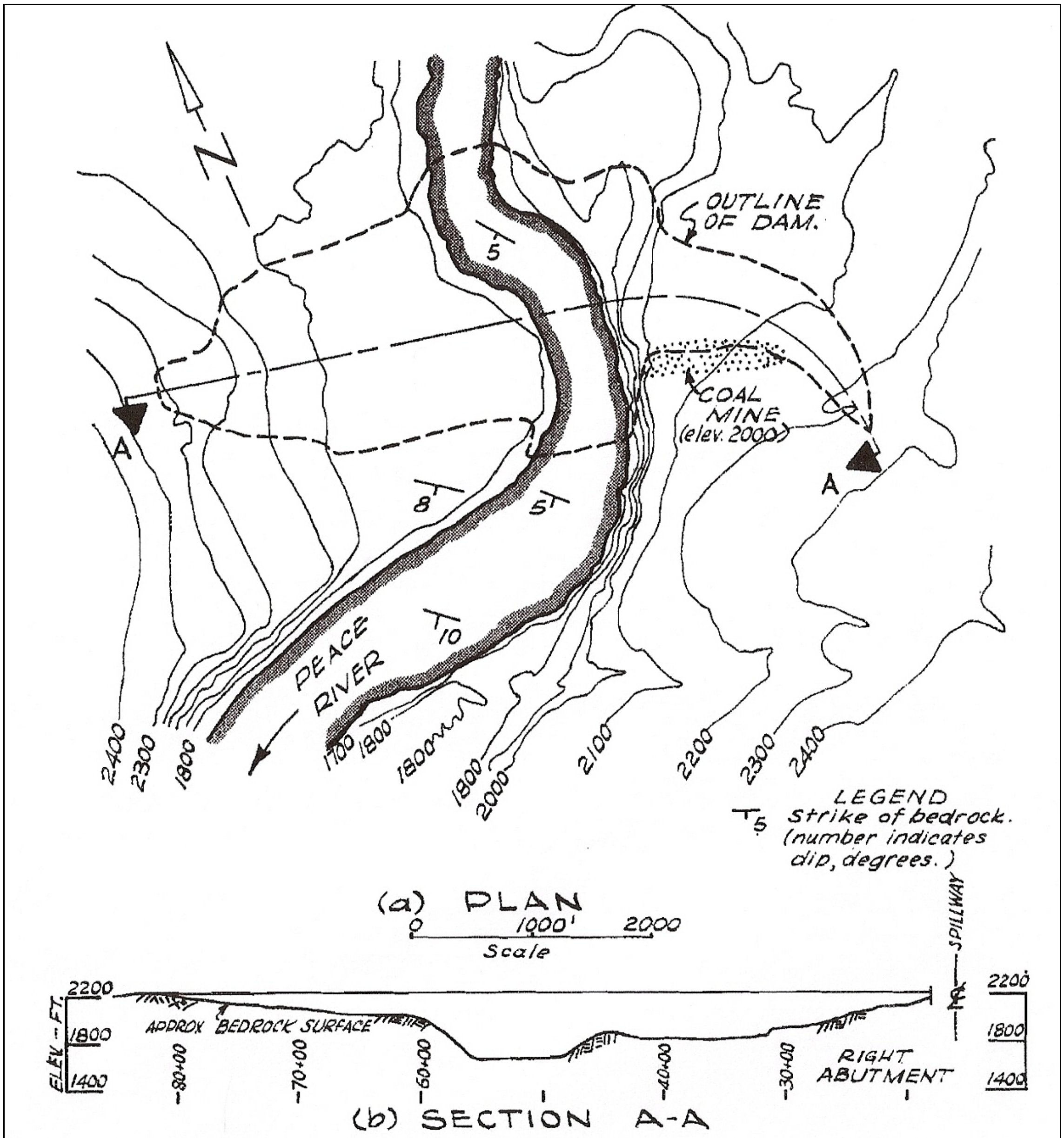
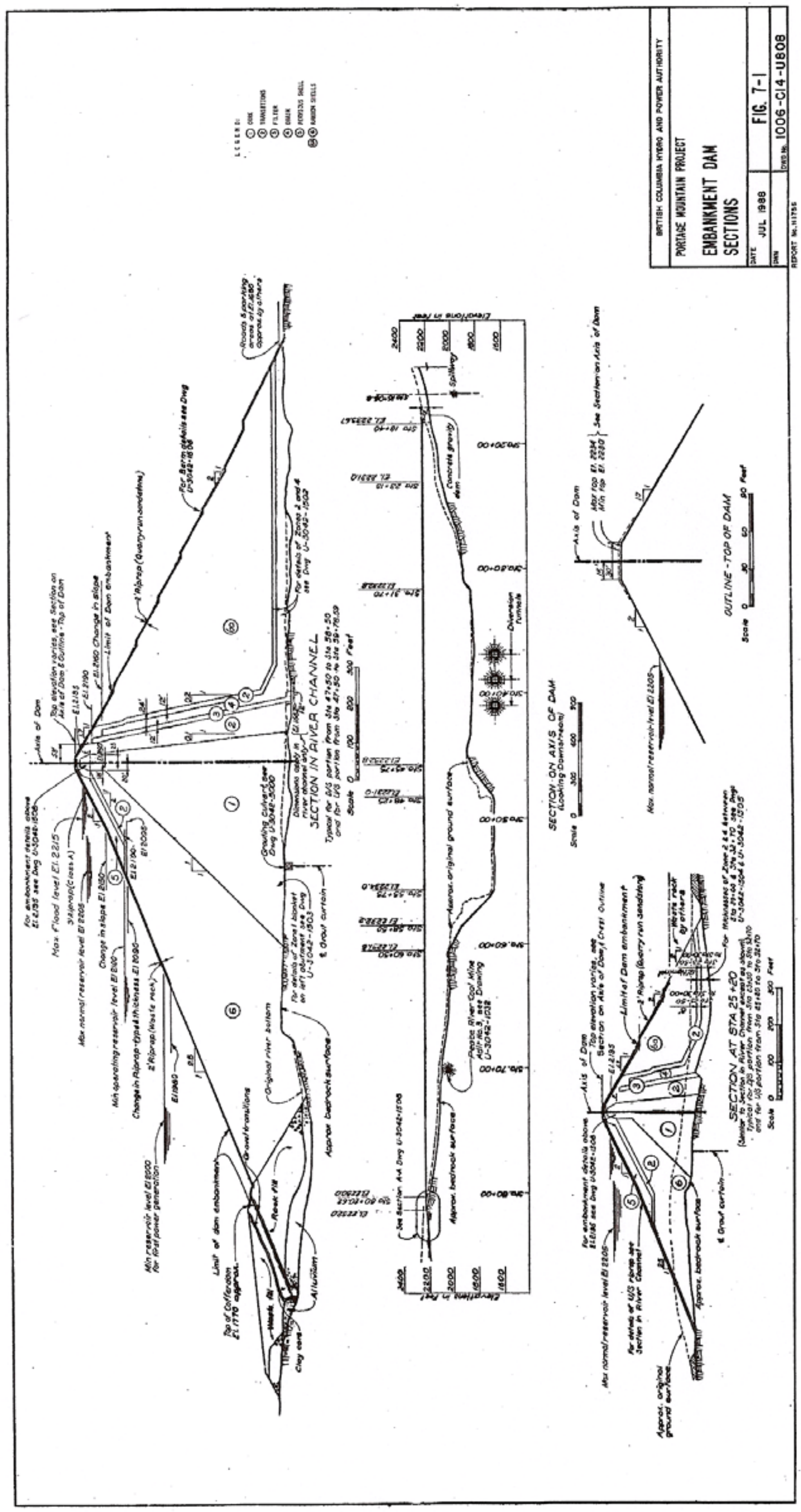


FIGURE 4. Damsite topography

C5 MATERIALS FOR DAM FILL

Deposits of lacustrine silts and clays, alluvial sands and gravels and glacial till were present in ‘unlimited quantities’ at locations close to the dam site. Two large terminal moraine deposits containing stratified sands and gravels, mostly in the size range 3-in to no 200 sieve (0.075 mm), with an average silt content of less than 10% were eventually selected as the sources of the various fills required to construct the dam, as follows:

- Shell: without processing
- Drainage zones: with minimal processing
- Core: with addition of about 10% fines.


BRITISH COLUMBIA HYDRO AND POWER AUTHORITY
PORTAGE MOUNTAIN PROJECT
EMBANKMENT DAM SECTIONS
FIG. 7-1
1006-C14-U808
SECTION IN RIVER CHANNEL
SECTION ON AXIS OF DAM
OUTLINE - TOP OF DAM
SECTION AT STA 25+20

C6 CROSS SECTION OF DAM

The dam section shown above (Fig 7-1 from Design Report H1756) was developed with three factors receiving particular attention:

- Stability of slopes
- Differential movements within the fill under its own weight and the full reservoir load
- Pattern of seepage through foundation and fill as a unit

The cross-section of the dam shown above includes steepened external slopes over the upper 80-ft of the dam, to 1 on 2 on the upstream slope (from 1 on 2.5) and to 1 on 1.7 (from 1 on 2) on the downstream slope. In two special areas the section was varied. The upstream slope was steepened near the powerhouse intake by using free-draining material in the upstream shoulder. At the right end of the dam, the downstream toe is buttressed by a toe fill to compensate an adverse dip of the bedrock surface.

In the discussion Ripley (1967) mentions that the dam cross-section had been modified progressively during construction to make best use of available materials. Figure 6 in Taylor (1969) shows the changes made to the dam section. One such modification seems to have been the inclusion of Zone 6 Shell fill in the 'key way' below Zone 1 Core, Zone 2 Transition and Zone 3 Filter on the left flank terrace.

C7 OVERBURDEN AND FOUNDATION TREATMENT

Overburden thickness at the dam site was 'insignificant'. On the canyon floor generally there was a scattering of large boulders but there was a 50-ft deep scour hole filled with sands, gravels and boulders over the upstream third. Most of the right abutment bench was covered with 5-10-ft of gravel-boulders. The left abutment bench was mantled in 15-20-ft of silt. The sloping bedrock surfaces at each end of the dam were covered by silt terraces, up to 100-ft thick.

Overburden was removed from the entire dam foundation, except at the deep scour hole in the canyon where only a lobe of uniform fine sand, thought to be susceptible to liquefaction was removed, and the alluvial sands and gravels below the sand were left in place.

C8 TREATMENT OF ROCK FOUNDATION

Foundation rock excavation was stated to be undertaken only in the Core contact area, including the shaping of the canyon walls described below and removal of up to 20-ft of weathered shale on the terraces above the canyon walls. However, it seems that the term 'Core contact' included the Core and filter system, as the construction records and later papers (see C32 below) show that foundation rock excavation was extended into the Transition and Filter contact zones downstream of the Core contact (see EEP Report sections 2.2.3 and 6.3.3 for more details). The sandstone exposed in the riverbed Core contact area was of hard unweathered flat plates and no excavation of unsuitable material was necessary.

Taylor (1969)[5] reported that excavation of weathered rock was as deep as 100-ft. He also noted that during excavation for the grouting culvert, a series of near vertical cracks were found in the riverbed foundations. Some were 12-14-in wide and extended to a depth of about 40-ft. It was thought that the cracks resulted from stress relief as the overlying rock load was removed by river erosion. The

[5] Taylor H (1969) WAC Bennett Dam. Eng'g Journal Canada, vol 52, no 10 October pp25-34

cracks were sealed with grout. Frequent vertical cracks were also found in the canyon walls, also thought to result from stress relief (Taylor & Morgan, 1970)[6].

The rock surface treatment in the Core contact area consisted of dental concrete, slush grout and gunite (see EEP Report sections 2.2.3 and 6.3.3 for more details of actual extent and local treatments). The dental concrete was used to smooth out major irregularities between bedding planes, primarily on the steep canyon slopes. The sandstone required treatment only with slush grout at occasional vertical joints. Gunite was used extensively on shale and coal surfaces to smooth out minor irregularities and to prevent weathering. Surface treatment of the foundation other than the Core Transition Filter contact areas was less extensive but all steps, discontinuities and weathering requiring treatment were identified by the geologists during mapping of the foundation and dealt with appropriately (see EEP Report sections 2.2.3 and 6.3.3 and Appendix D for more details).

C9 TREATMENT AND FLATTENING OF CANYON SLOPES

Following studies (Taylor, 1969 & Taylor & Morgan, 1970), to avoid differential strains in this high dam, sharp discontinuities the steep canyon walls in the Core contact area (including Transition and Filter contact areas) were rounded off and slopes were flattened to 1 on 1 at depth in the canyon and 1 on 2 near the top. This is shown in the section on the dam axis on the figure (Figure 7-1 from Design Report H1756) below and there are more detailed views in Taylor & Morgan, 1970 (and on Figures 6, 7 and 8 in the EEP Report). The 'Core contact' area also converges slightly downstream to reduce the 'gradient of differential movements caused by water load and create a desirable wedge action'. The fill materials are of low compressibility and it was noted that the characteristics of all the fills would be similar as they are derived from similar materials.

C10 CONTROL OF SEEPAGE THROUGH THE FOUNDATION

To avoid local areas of concentrated seepage and high hydraulic gradients and recognizing that seepage in the foundation would largely be through bedding planes and infrequent joints, the following measures were included in the dam and foundation:

- Wide contact area between Core and bedrock
- Blanket grouting beneath the Core contact area to a depth of about 20-ft
- Multi-row grout curtain, five lines generally and three lines at the abutments, to variable depth averaging about 350-ft. The closure criterion was that water loss during a water pressure test in a 3-inch hole should not be more than one Lugeon unit (Taylor, 1969).
- Reasonable but not excessive foundation drainage provided through a blanket drain below the downstream shell, deep drainage tunnels about 200-ft downstream of the grout curtain, with a curtain of grout holes drilled both upwards and downwards from the tunnels, and tunnels to the downstream toe to release drainage into the seepage measurement chambers. Drainage holes 3-in in dia and from 30 to 300-ft long were also drilled at 50-ft centres upward and downwards from the tunnels to drain the downstream side of the grout curtains. There were no drainage tunnels below the old river channel, only two inclined holes drilled from the ends of the tunnels (Taylor, 1969).

[6] Taylor H & Morgan G C (1970) Measures taken to limit the possible development of cracks in a high earthfill dam. ICOLD 10th Congress Montreal Q36 R40 pp679-702

C11 CONTROL OF SEEPAGE THROUGH THE DAM

To provide favourable seepage conditions and to avoid high gradients in the dam fill, the following principles were applied:

- Core width equal to water depth
- Wide 'transitions' between Core and 'internal drain'. The 'upstream transition', Zone 2, (later called 'Transition') varies in width from 80-ft at the base of the dam to 33-ft near the top; the 'second transition', Zone 3 (called 'Filter' later), has a uniform width of 12-ft. The 'Drain', Zone 4, also has a uniform width of 24-ft (but is deeper in the blanket over the foundation).
- Reasonably uniform increase in permeability from zone to zone going downstream. Typical values of fill placed to 1967 was about 10^{-6} cm/sec in Zone 1 Core, about 10^{-4} cm/sec in Zone 2 Transition, about 10^{-3} cm/sec in Zone 3 Filter, and about 10 cm/sec in the Zone 4 Drain. The permeability of the random Shell, Zone 6, was about 10^{-5} cm/sec.
- Migration tests were carried out (Taylor, 1969) to examine the filtering capacity of the Filter against the Transition (it appears). If the Filter contained no particles between ⅜-in (10 mm) and #10 sieves (2 mm), about 25% of the Transition was lost. When about 30% of the Filter comprised 10 mm to 2 mm particles the loss was less than 5%.

C12 MEASURES TO LIMIT CRACKING OF THE CORE

Taylor & Morgan (1970) were particularly concerned about cracking of the Core resulting from differential settlement. They listed the following measures taken to enable the dam to withstand some cracking of the Core should it develop:

- Availability of abundant fine material to provide a wide Core zone
- Core material was well-graded and non-plastic, and although cracks may develop in such materials, they tend to seal themselves readily as water seeps through the cracks
- Shell materials upstream of the Core contain appreciable fines and sand content which may also wash in and seal any cracks in Core
- Thick Transition and Filter zones, controlled gradings by use of processed materials, avoiding segregation and sharp gradation changes
- Drainage zone isolated the downstream Shell supporting the Core from any concentrated seepage such as could develop from cracking.

C13 STABILITY AND SEISMIC STABILITY

The south moraine was the main source of fills (Morgan & Harris, 1967)[7]. It contained sandy gravels and sands. Concerns about susceptibility to earthquake shaking led to the specification not permitting the use of uniform fine grained granular materials. The prohibition was considered particularly important upstream of the Core and in the more highly stressed portions of the dam.

The dam was designed, largely on the basis of considerable judgment given the understanding of earthquake phenomena at the time (1967), to withstand a major earthquake. Taylor (1969) gives details of the stability analyses including seismic stability. Stability analyses for rapid drawdown and steady seepage were completed, both with and without earthquake. The earthquake factor used was 0.1**g**. As negligible pore pressures developed during construction, the 'end of construction' case was

[7] Morgan G C & Harris M C (1967) Portage Mountain Dam II. Materials. Canadian Geotechnical Journal, vol IV, no 2 (on CD1 March 2011).

not considered. The rapid drawdown condition assumed no drainage and was thought to be extremely conservative for the highly dilatant materials used in the dam and as drawdown rates of 2-in/day were the maximum possible. Lowest factors of safety (Table 1, Taylor, 1969) were 1.4 for static rapid drawdown at sta 63 (near the intakes), 1.9 in the canyon; and 1.0 in earthquake conditions at the intakes and 1.5 in the canyon.

C14 PROCESSING OF FILL MATERIALS

Materials from the south moraine and elsewhere were extensively processed to achieve the stringent specifications for fill in this high dam. Processing also reduced wastage of materials, for example by making it possible to mix the otherwise excluded uniform fine grained granular materials with others. It reduced variability in gradation and eliminated the possibility of segregation that can occur in Core materials containing large sizes. Processing also made it possible to add non-plastic silt from a source close to the processing plant to the Core material. About 44% of the fill used in the dam was processed (Taylor & Morgan, 1970). The Zone 4 Drain and foundation drainage blanket downstream of the Core, Transition and Filter carries away seepage water and thereby allowed the placement of all well-graded materials occurring naturally within the moraine, or as excess from the processing operations, in the Zone 6 downstream Shell fill.

C15 COMPACTION TRIALS

Compaction trials were conducted to determine densities achieved for varying numbers of passes of pre-selected compaction equipment and layer depths. Zone 1 Core layers were 10-inch and Zone 6 Shell layers 15-inch, both post-compaction. Optimum moisture contents for both Core and Shell were from 6% to 8%. There was little difference in densities achieved by more than two passes of 100-ton pneumatic compactors on the Core and two 6-ton vibratory rollers pulled in tandem on the Shell. It was found that the density of fill placed in the trials was greater than that in the dam. This was thought to be because the dam fill contained more of the finer material than the trial fill because it was mined from deeper in the moraine. Table 1 below shows the fill densities attained.

C16 ZONE 6 SHELL FILL

The Zone 6 Shell fill, which was predominantly as-dug material from the moraine, was essentially well-graded when viewed on average, but with a deficiency of particle sizes between 4.75 mm and 2 mm (no 4 to no 10 sieve). This deficiency made it difficult to process the moraine to produce Transition, Filter and Drain materials within specification because there was a tendency for gradations to be too coarse on the ⅜ inch (10 mm) sieve and too fine on the no 4 (4.75 mm) sieve.

C17 PERMEABILITY OF FILLS

The permeability of the moraine measured in-situ from surface test pits was about 10^{-3} cm/sec. Permeability of the Core was measured in triaxial consolidation tests and 8-in dia permeameters. Permeabilities of the other fills were measured in 22-in dia ‘gas barrel’ permeameters. Shell permeability was also determined from triaxial test results. The results are shown in Table 1 below.

The permeabilities of the dam fills and the foundations are examined further in Chapter 2 and Appendices D and E of the EEP Report.

TABLE 1

Typical soil properties of Portage Mountain dam based on field and laboratory testing to June 1966

		As placed				Shear strength		
						Total stress		Effective stress
Zone	Soil type	Field dry density lb./cu. ft.	Field moisture content %	Maximum laboratory dry density lb./cu. ft.*	Estimated average permeability cm./sec.	c psi	ϕ	ϕ'† ($c' = 0$)
Core (1)	Silty-sand (SM)	128	6.9	130	2×10^{-6}	31	15°	37°
Transition (2)	Gravelly-sand (GW)	133	6.1	138	1×10^{-4}	38	28°	38°
Filter (3)	Sandy-gravel (GP)	136	4.0	137	1×10^{-3}	38	28°	38°
Drain (4)	Gravel (GP)	114	1.6	114.5	10 (not tested)	0	36°	36°
Pervious shell‡ (5)	Sandy-gravel to gravel (GP)	110	2	115	10	0	36°	36°
Random shell (6)	Sandy-gravel (GW-GP)	135	6.1	140	2×10^{-5}	38	28°	38°

*Core uses impact compaction (ASTM D 698, 6 in. mould), all other zones use vibratory compaction (USBR Method E 12 B).
†Apparent cohesion neglected.
‡Assumed properties, no material placed to date.

C18 SHEAR STRENGTH OF FILLS

The results of triaxial tests on 6-in and 12-in dia samples are also shown in Table 1 above. They indicate slightly higher ø' of Shell material than Core. All tests showed a reduction in shear strength after peak. Tests conducted on low density materials showed a tendency to peak early. The tests showed the desirability of adequately compacting the Core material. Every effort was to be made to exclude low density soils from the Core.

C19 PARTICLE BREAKDOWN DURING TESTING

Particle breakdown was observed during consolidation and shearing of many of the tests. For Core material the consequence was an increase in fines of 1-2% passing the no 200 (75 micron) sieve with cell pressure of 400 lb/sq in (about 2,750 kPa). In the Shell material the breakdown was markedly greater at 17% on the no 4 (4.75 mm) sieve and about 2% on the no 200 (75 micron) sieve.

C20 COMPRESSIBILITY OF FILLS

The fill materials are of low compressibility (preliminary observations indicated moduli of 30,000 to 70,000 lb/sq in for Core and 40,000 to 100,000 lb/sq in for Shell). It was noted that the characteristics of all the fills would be similar as they are derived from similar materials. The measured compressibility was very low. Laboratory values of m_v ranged from 0.2×10^{-6} cm^2/gm to 7×10^{-6} cm^2/gm for very dense Shell (142 lb/cu ft) and low density Core (118 lb/cu ft) samples. Values from settlement records to July 1966 when the maximum fill depth was 320-ft gave values of 0.2×10^{-6} cm^2/gm to 1.0×10^{-6} cm^2/gm in the Core and 0.07×10^{-6} cm^2/gm to 0.3×10^{-6} cm^2/gm, leading to predictions of post-construction settlement of less than one foot. It appeared that fill compression occurred almost immediately after placing.

C21 CRACKING OF FILL AT CANYON EDGES

Taylor (1969) discusses the settlement and the ‘stretch’ that may cause cracks where the fill depth varies sharply at the canyon edges. The widths of Filter and Drain were conservative to maintain capacity if cracks occurred and it was also noted that seepage through a crack would tend to cause plugging.

C22 PORE PRESSURE RESPONSE IN FILL

Observations of pore pressure response to fill loading showed a response of less than 5% of applied stress. The degree of saturation of the fill was 65% or less.

C23 CHANGES TO SPECIFICATIONS

Changes were made to specifications and designs in the light of experience. The Zone 3 Filter tended to be gap-graded with variable gradation, and occasionally exceeded the specified fine limit. The processing procedures were adjusted to rectify this, but the Filter remained slightly gap-graded due to the shortage of coarse sand sizes in the moraine itself, although in other respects it was thought acceptable.

C24 ZONE 5 RIP-RAP

The Zone 5 rip-rap wave protection on the upper part of the upstream face was quarried sandstone with D_{50} size of 24-in, placed in a 3-ft thick layer down to 2,100-ft, the minimum operating level (Taylor, 1969). For 100-ft below the rip-rap, the slope was protected by a 2-ft layer of waste rock. No protection was provided at lower levels, well below any likely operating level. The downstream slope was coated with a 2-ft layer of quarry-run sandstone.

C25 CHANGES IN DAM DIMENSIONS AT CREST

Changes were made to the upstream pervious zone (Zone 5) and the adjacent Transition zone (Zone 2). The base of the Zone 5 was raised from 2000-ft to 2100-ft elevation, and the thickness of the Zone 2 Transition below it was much reduced, with the difference being made up with unprocessed random Shell Zone 6 material, justified by the fact that the properties of the Transition did not differ appreciably from those of the Shell.

As mentioned earlier, the upstream slope of the part of dam less than 200-ft high was steepened from 1 on 2.5 to 1 on 2 to shorten the length of the penstocks.

The crest of the whole dam was also altered. The crest width was reduced from 40-ft to 30-ft, and the upstream slope steepened from 1 on 2.5 to 1 on 2 above elevation 2150-ft, and the downstream slope from 1 on 2 to 1 on 1.7 above 2160-ft. Crest level is about 2230-ft. The changes are explained by Taylor (1969) and shown in his Figure 6. The changes made savings by reducing the amounts of fill by about 3 million cu yds and were justified by better than expected fill properties, the actual Core ø' value of 37° compared to 30-34° assumed, the very limited pore pressure response to loading, and the very satisfactory field experience; the contractor’s blending operation had been generally quite acceptable and placement and compaction presented few problems.

C26 REDUCTION IN ZONE 5 DIMENSIONS AT CREST

Another significant change was also made to the upper parts of the dam. It was found that the material remaining in the moraine was finer than that obtained previously. As the final 200-ft of the dam required a large volume of coarse, processed material, mainly for the large free-draining zone on the upstream side of the dam, the stability of a narrower (40-ft wide) free-draining zone was examined and found to be acceptable (Taylor, 1969, Figure 8). The fine moraine material could be used to substitute for the reduced width of the coarse Zone 5 material, and it was found that the inclusion of finer Shell material in the upper part of the downstream Shell had no adverse effects. The reduction in the Zone 5 material volume also made substantial savings.

C27 PRECAUTION AGAINST RAPID DRAWDOWN FAILURE

To provide some drainage capacity in the relatively fine upstream Shell fill Zone 6 as a precaution against rapid drawdown failure, the fill was placed in upstream-downstream layers, so that the more permeable 'streaks' between layers provided an outlet for pore water.

C28 RATE OF CONSTRUCTION AND FROST PENETRATION

Construction control was described by Low and Lyell (1967)[8]. Fill placing could proceed only in the summer months, between mid-April and early November. Penetration of frost in the fill to depths of 8-ft has been recorded. There was little rain in summer; construction was interrupted by rain on only five days in 1965. Fill placing at rates of 2.5 million cu yds per month was required. In 1965, over 3 million cu yds per month were placed, with a peak of 141,690 cu yds in one day. The high rates were achieved by the construction methods adopted by the contractor, which included the use of conveyor belts 3 miles long to transport materials from the moraine source to the processing plant and the dam. They were facilitated by the control methods developed by the consulting engineer.

C29 METHOD-BASED SPECIFICATIONS

The embankment construction specifications were method related, not end product based. The contractor was responsible for providing fills of the correct gradation and placing them to the methods specified (see overleaf for Table 1 from Low & Lyell). The engineer took samples to confirm that the methods specified were producing the quality of fill required. Compactors were paid for by the hour, subsequently compactive effort could be varied readily from the method specification if necessary.

C30 TRANSPORT AND PROCESSING OF FILL

Material from the moraine was first fed to a dry screening system and split on ⅜-inch and 1½-inch screens. The coarse 1½-inch fraction was washed for use in Filter and Transition zone materials. The fine ⅜-inch fraction (which was unwashed sand) was used for blending as Core material. Some of the fine fraction and the washings from the coarse fraction were used for blending in the Transition and Filter zones.

C31 SILT FOR BLENDING TO PRODUCE CORE FILL

The silt for blending with the unwashed sand to make the Core material was hauled from a source near the processing plant. The silt was glacially laid varved silt (Long & Smith, 1967)[9]. The quantity passing a #200 (0.075 mm) mesh had been known to fluctuate from 25% to 99.9% in a matter of hours. The inclusion of this variable product with dry screened sand to make Core required nearly continuous adjustment of reclaim control gates.

The difficulties are confirmed in the 1965 Report on Fill Placement Quality Control[10] which states that it was possible to see the sand and silt separately, particularly after rain. To facilitate easier field blending it was decided in late August 1965 to reduce the compacted Core, Zone 1, layer depth to 6-inches (from 10-inches). It is not clear if the 6-inch layers were continued in the subsequent fill placing years.

[8] Low W I & Lyell A P (1967) Portage Mountain Dam III. Development of construction control. Canadian Geotechnical Journal, vol IV, no 2 (on CD1 March 2011).
[9] Long D H & Smith G A (1967) In discussion of Low & Lyell (1967)
[10] Fill Placement Quality Control Analysis of Field Tests for 1965 (on CD 2 March 2011)

C32 BLENDING OF MORAINE MATERIALS FOR FILL

The unwashed gravel and the three types of sand could be combined in the blends required for the dam fills and filters. Gradations of the moraine source, the blends and the various fills and filters are shown below on Figure 5 from Low & Lyell.

Long & Smith (1967) highlight the difficulties in making efficient use of the moraine materials to meet the demand for fills in the sequence and quantities that the dam profile, and the level fill surfaces, imposed.

TABLE I. Summary of embankment placing specifications

ZONE		GRADATION		MOISTURE CONTENT			LAYERS				ROLLER			SPECIAL REQUIREMENTS
		Screen Size	Limits % Passing	LIMITS	Supplementary Conditioning	Restrictions no placing when —	Max. placed Thickness	Lateral Overlap	Slope	Elevation Difference	Type	Coverages	Speed	
CORE	1	3/4" #60 #200	100 35-60 25-35	From designated increased at abutments, foundations and concrete surfaces + 1% − 2%	At loading bins. Requires mixing if added on embankment	1. Freezes 2. Rain 3. Snow or sleet	10"	2 ft. at Zone 2 5 ft. at Zone 6	2% away from 2	2ft. higher at abutments Max. 2.5 ft to adjacent Zone	Pneumatic 45 Kipwheel load and 100 p.s.i. Minimum	4	5 m.p.h.	Spread parallel to Dam Axis. Scarify preceding layer Max. 3". Free from lenses, pockets, streaks and layers of differing material
TRANSITIONS	2	1 1/2" #60 #200 Max. Size 6"	80-100 5-30 0-5	± 2 %	On embankment	Do	20"	1 ft. at Zone 1, 5 and 6.	3-5% away from 3	Maintain surface above Zones 1 & 6 Max. 2.5 ft. to adjacent Zone	Vibratory Min. 12,000 lbs. 1100 to 2400 vib/min.	4	2 m.p.h.	Spread parallel to Dam Axis. Scarify preceding layer. Free from lenses, pockets, streaks and layers of segregated material. N.B. Max. Size 6"
FILTER	3	3" #40 #100	100 0-10 0-3	Min. 5%	On embankment	Surface freezes	20"	Max. reduction of Zone width is 1.0 ft.	3-5 % away from 4	Do	Vibratory	2	2 m.p.h.	Spread parallel to Dam Axis. Free from lenses, pockets, streaks and layers of segregated material.
DRAIN	4	3" #4 #40 Max. Size 4"	90-100 0-40 0-2				20"	Max. reduction of Zone 4 & 2 2ft.	To be highest point on fill	Maintain surface above Zones 1 & 6 Highest Zone Max. 2.5 ft. to adjacent Zone	Vibratory	2	2 m.p.h.	Spread parallel to Dam Axis. Free from lenses, pockets, streaks and layers of segregated material. N.B. Max. Size 6"
PERVIOUS SHELLS	SELECT	3" #4 #40	75-100 0-40 0-5				20"	2 ft. at Zone 2	3-5 % away from 2		Vibratory	2	2 m.p.h.	Spread parallel to Dam Axis.
	OTHER	3" #4 #40 Max. Size 18"	Max. % Passing 100 40 5											N.B. Max. Size 18"
RANDOM SHELLS	6	3" #20 #200 Max. Size Layer thickness	60-100 0-45 0-12	± 2%	Minor final correction on embankment	As Zone 1	15" general	2 ft. at Zone 1 upstream and Zone 2 downstream	3 - 5% away from 1 & 2		Vibratory	2	2 m.p.h.	Dump and spread perpendicular to Dam Axis Streaks and layers permissable. Rock pieces larger than layer thickness in outer 50ft.

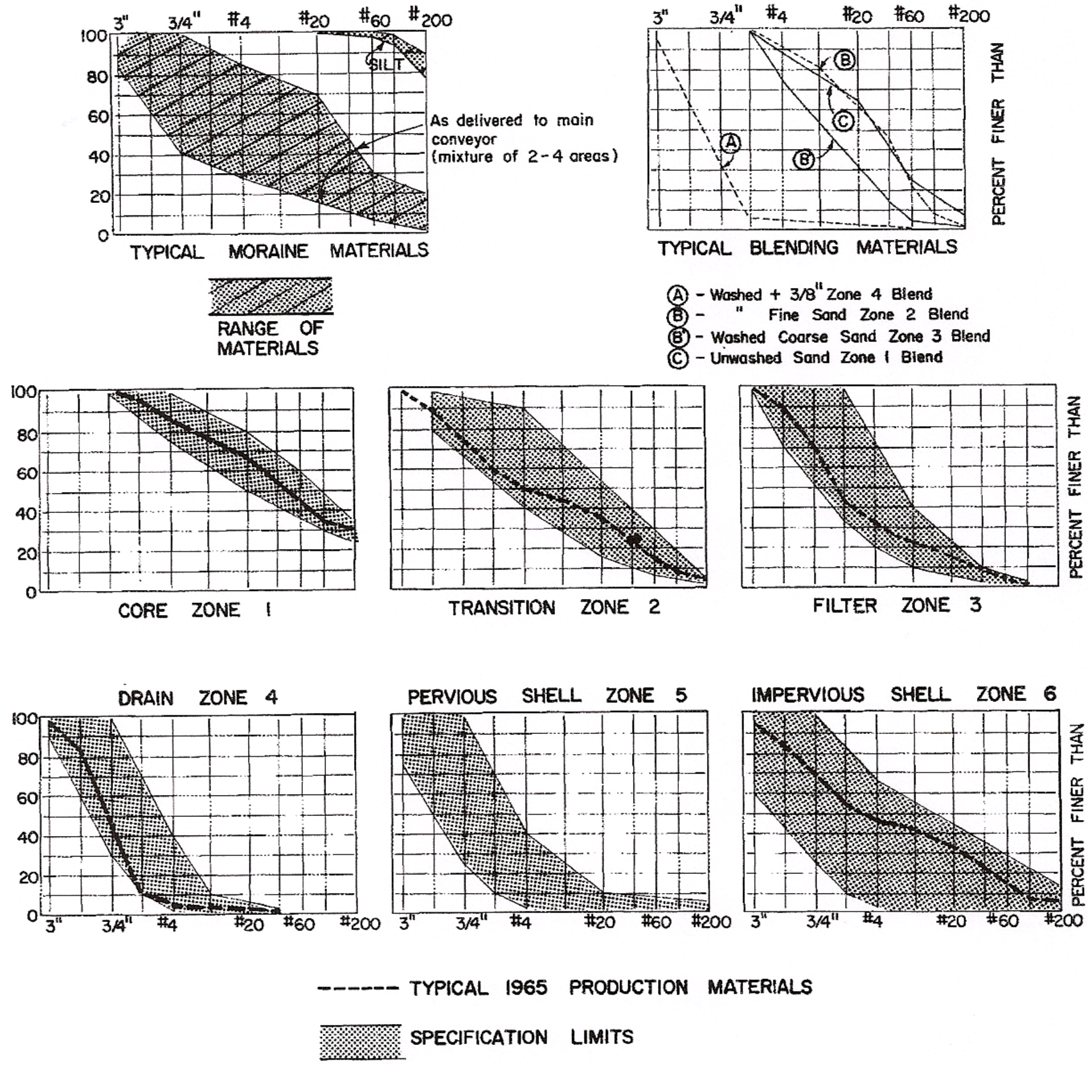


FIGURE 5. Typical moraine, blending, and zone materials

C33 FOUNDATION TREATMENT

'After shaping of the Core contact by excavation, and prior to blanket grouting, the major part of the contact surface below the Core and below the Transition and Filter zones was either shaped with concrete fillets or protected by pneumatically applied mortar'. The shaping criteria are described above from Ripley (1967). Areas not treated in this manner were generally smooth and sound sandstone rock. After completion of the blanket grouting these areas were cleaned by hosing with water if necessary, and by air to remove all loose material. This is shown on Figure 7 of Low & Lyell, 1967. The pneumatically applied mortar was then inspected and all loose sections were removed and repaired if necessary'.

'The areas up- and down-stream of the Core, Transition and Filter zones were stripped of all overburden and machine cleaned, using Caterpillar D8 bulldozers and front-end loaders. A more careful machine clean was required downstream of the Core to prevent contamination of the drainage blanket. The final result in this area was generally a rock surface with a thin film of fines about ½-inch thick. Under the drainage blanket in the canyon area, where wet and muddy pockets were encountered, loads of moraine gravel were dumped to mop up the moisture. This material was then bulldozed away immediately ahead of the Drain gravel'.

C34 PLACING OF FILL ON PREPARED FOUNDATION

'Initial placing of the Core on the smooth undulating rock in the river bottom was done by end-dumping, spreading with a D4 bulldozer, and compacting in very limited areas by loaded Euclid end-dump trucks. Material was compacted in very limited areas against rock faces by 950 lb Terrapac CM20 vibrating plate hand-guided compactors. The Core contact at an early stage is shown in Figure 8. As soon as space permitted, delivery of Core material with Athey 100 ton bottom-dump trucks was started and the 90 ton pneumatic-tired roller used for compaction'. As the fill area increased the normal construction sequence of dumping in windrows from bottom dumps, spreading by Caterpillar 834 rubber tired bulldozers, levelling by Beegee land leveller towed by a Caterpillar 660 rubber tired unit, and rolling with a 90 ton pneumatic tired roller was developed. 'Two passes of the roller were considered as one coverage'.

C35 COMPACTION AT CORE CONTACT WITH ABUTMENTS

At the Core contact with the abutments, a 2-ft wide zone was compacted in 6-in lifts using a twin-drum Bomag BW75 hand-guided self-propelled vibrating rollers. No increase in moisture content was used because the Core is non-plastic and would not flow into any indentations, and experience had shown that the material became sloppy at moisture contents above optimum.

Moisture conditioning of the Core was done at the blender or along the final delivery belt.

C36 PLACING, SPREADING, LEVELLING, SCARIFYING AND COMPACTING FILLS

Core, Transition, Filter and Drain zones were placed parallel to the dam axis, but both upstream and downstream Shells were placed in layers perpendicular to the axis with the intention of providing drainage along the more permeable segregated material at the edges of the placed layers. In the event, Low & Lyell report that it was found that the Shell material did not segregate on placing, and its characteristics were such that drainage was not necessary.

All zones were delivered, spread and levelled similarly. Zones 1 (Core) and 2 (Transition) were scarified to a depth of about 2-in between lifts. Zones other than the Core were compacted with 6 ton Ferguson vibrating rollers (drum weight 12,000 lbs, 1,100 to 1,200 vibrations per minute, Taylor 1969). The contractor used three tandem pairs towed by a D6 bulldozer. One pass of this arrangement was considered as two coverages. Compaction of the abutment contacts of the Filter and Transition zones was done as described above for the Core. No special compaction was done at the contact of the Shells with the rock foundation, other than running the rollers as close to the rock as possible.

C37 INSTALLATION OF IINSTRUMENTS

Instruments were installed as work progressed. The cross-arm settlement gauges and the observation wells were normally kept buried at least 18-in below the fill surface to limit obstacles in the path of the 100 ton bottom dump trucks. When the instruments were about 7-8-ft below the surface, a trench was excavated down to them and the instrument extended. The trenches were backfilled using hand-held air-driven tamper to cover the instrument. Periodic density tests showed that the hand compacted fill was 'comparable in density to the main body of the fill'.

The piezometer lines were buried in near horizontal trenches in the fill. Some lines were extended vertically through the fill; these were called 'risers'. Two risers, each comprising six ¼-in ID pvc tubes were embedded in the fill without a surrounding casing. To prevent damage, all piezometer lines were embedded in material passing a no 4 (4.75 mm) sieve. In the risers the <#4 material was fed through a small timber hopper which also served as a guide frame for the lines. This surrounded the tubes by about 2-in of the <#4 material. The fill outside the hopper was compacted with a hand-guided vibrating roller, as was the fill in the horizontal trenches. At the riser locations the cross-arm device and the observation well were also kept above the fill. The piezometers were led to corrugated metal terminal houses embedded in the fill at the downstream slope. The terminals were bedded on <⅜-in (10 mm) sand and normal Shell fill was compacted around them with hand-guided rollers.

C38 PNEUMATIC ROLLER TYRE PRESSURES

A tyre pressure of 100 lb/sq in was initially selected for the pneumatic tired roller used to compact the Core. This was used in 1964 and until mid-1965 when it was increased to 130 lb/sq in to alleviate tyre trouble. The 90 ton roller applied 45,000 lb wheel load (Taylor, 1969).

C39 FIELD TESTS FFOR DENSITY AND OTHER FILL PROPERTIES

The field density of the Core was determined by the standard sand cone test (ASTM D1556). The 6-in dia test was large enough as all Core material passed the no 10 (2 mm screen). The hole excavated to a depth of between 7 and 8-inches. The layer depth was 10-in.

Field density of the other fills, all of which are gravel, was done by placing a 30-inch steel template on the fill surface and excavating to a depth of 15-in to remove about 150 lbs of material. The hole volume was determined by lining it with 2 mil polyethylene sheet and filling with water.

Other field tests included pits to examine fill quality, zone contacts and density profiles and permeability tests using the well method. Lift thicknesses were also checked.

The density specified for the Core was at standard Proctor compactive effort. This was chosen because the optimum moisture content and maximum density achieved closely matched the roller compaction curve determined in the field. Field densities were checked for compliance by comparing the field density to the laboratory density of the collected sample re-compacted at the estimated optimum moisture content in the laboratory. This approach was adopted because the variation in optimum density from variations in gradation was greater than the variation between moisture content.

The maximum density of the other fills was determined by the USBR vibratory apparatus (ASTM D2049-64T). The field density was compared to the laboratory value of the maximum density of each sample, taken as 100%.

Samples for grading were dried in gas-fired drying pans rather than ovens because of the many samples involved. There was no loss of accuracy because no clays or organic materials were used in the dam.

A high capacity consolidometer was used to measure Core compressibility and to conduct falling head permeability tests on the Core material under confining pressures.

Typical frequencies of tests were as follows:

Zone	Field density (cu yds per test)	Gradation (cu yds per test
1, Core	4,000	1,500
2, Transition	5,000	1,800
3, Filter	2,500	500
4, Drain	50,000	3,000
6, Shells	10,000	3,500

C40 FIELD DENSITY STANDARDS

The density standards are shown in Table III below from Low & Lyell. Note that the Core density is relative to standard Proctor compactive effort. Densities of the other fills are relative to maximum vibrated density.

TABLE III

Fill density standards

Zone	Minimum acceptable	Minimum desirable average	Maximum percentage of tests below min. acceptable
Core (1)	95%*	98%*	10%
Typical	125 lb./cu. ft.	128 lb./cu. ft.	
Transition (2)	90%†	95%†	10%
Typical	125 lb./cu. ft.	130 lb./cu. ft.	
Filter (3)	92%†	96%†	10%
Typical	128 lb./cu. ft.	133 lb./cu. ft.	
Drain (4)	92%†	98%†	10%
Typical	112 lb./cu. ft.	118 lb./cu. ft.	
Random shell (6)	90%†	95%†	10%
Coarse	105 lb./cu. ft.	108 lb./cu. ft.	
Average	135 lb./cu. ft.	139 lb./cu. ft.	
Fine	125 lb./cu. ft.	129 lb./cu. ft.	

*Based on ASTM Designation D 698-58T D
†Based on USBR Designation E. 12 B.

C41 STANDARDS ACHIEVED

The standards set were surpassed in all fills except the Shells where the standard set was based on well-graded material but the fill placed was gap-graded with little material between #4 and #20 sieves (4.75 mm and approximately 1 mm) and a large amount between the #20 and #60 sieves (1 mm and approximately 0.2 mm). This proved difficult to compact to the densities required. Various alternative methods were tested without success, but as the actual in-situ dry density was seldom less than 125 lb/cu ft, it was decided to allow a minimum acceptable value of 90% of standard density.

Typical compaction results for the Core range from 98.2% to 100%. Typical Shell results were 94.3% to 98.2%. Other fills' results were between 95% and 100%.

Close control was achieved on site with laboratory and field teams working all 3 shifts per day with routine daily reporting and summary reports compiled and issued twice monthly.

C42 MAKING ZONE 6 SHELL FILL COARSER

Maintaining a balance of available materials to blend for the dam fills frequently led to the Shell material containing too much fine material. In these circumstances gravel 'add-rock' was incorporated into the Shell by spreading gravel over the surface and working it in using a 50-in dia disc plough capable of penetrating 18-in.

C43 CORRECTING CORE SILT CONTENT AND LAYER DEPTH

The Core material, a blended product, was less variable, but was sometimes poorly blended or had inadequate silt content. Low silt content could not be detected by eye and many gradation tests were necessary. Field blending using harrows and a 28-in dia disc plough capable of penetrating to 12-in were used to work in additional silt. The change of layer depth to 6-inches in late August 1965 (C30 above) is not mentioned.

There were concerns about thick lifts and inadequate compaction of Core placed initially in the narrow canyon section.

C44 SEGREGATION OF FILTER

The Zone 3 Filter was a blended granular material and proved difficult to manufacture and place without excessive segregation. Initially the Filter contained an excess of less than #40 (0.42 mm) sizes. In 1964 a single blending sand was used for both Transition and Filter zones, but early in 1965 a limited quantity of coarser blending sand was produced for use in the Filter only. The tendency to segregate continued and was corrected by discs and harrows. Filter containing excess fines was rejected, but rejected material was often suitable for Transition.

C45 PLACING ZONE 4 DRAIN

The Drain material, the washed product of material retained on the ⅜-in (10 mm) screens, sometimes segregated on dumping and spreading. Particular care was taken with Drain material placed in the near vertical 'chimney' Drain to limit segregation. Coarser materials were placed in the 'downstream wrap-around section of the Drain'.

C46 FROST PENETRATION AND PLACING ON FROZEN SURFACES

Tests showed up to 6-ft of frost penetration in the Core over winter, but very little heave occurred (in the order of 0.01-ft). Attempts to insulate and protect the fill surfaces were abandoned after the winter of 1964-65. After winter, Core filling was not re-started until it had dried out but filling on the Shells commenced provided that at least 12-in of the surface had thawed. Placing in cold conditions was permitted provided the temperature of the fill material being placed was above freezing. In Shell zones fill was placed on frozen surfaces. On the Core, placing was permitted on surfaces frozen to only ½-in deep provided the frozen layer was first removed and the temperatures were high enough to allow compaction without freezing.

C47 SETTLEMENT OF FILL AND FOUNDATION

Taylor (1969) reports that settlement from the observation wells and cross arm gauges was 1.1-ft maximum, about 90% had occurred during fill placement. Horizontal movements were small, a maximum of about 2-in into the canyon at the canyon walls.

Taylor & Morgan (1970) add to the settlement information. The fill at mid-height moved down 12-in (31 cm) relative to bedrock, the bedrock settled about 25 cm (8-in) in grouted zones and 31 cm (12-in) in ungrouted ones. Surface settlement since completion had been about 1-in (3 cm). The compression was about 0.15% strain in the Core and 0.1% in the Shell. Downstream movements from water load were between 2 and 3-in.

C48 EARLY PORE PRESSURES AND SEEPAGE IN DAM AND FOUNDATION

Taylor (1969) presents some results on the early performance of the dam to May 1969, when the water depth was 400-ft (to elevation 2080). Pore pressures dropped markedly across the Core, Transition and Filter to be almost at foundation level at the Drain. Pore pressures in the deep foundation were high but well below overburden load. Seepage quantities from all seven stations in the tunnels were very low totalling 0.2 cu ft/sec. No seepage had reached the collection system at the downstream toe of the dam. It was thought that any seepage reaching this area had passed through fractured rock to drain into the tailrace channel, which is at a lower elevation.

Taylor and Chow (1976)[11] report on the development of seepage flows and pore pressures on first filling of the reservoir. There was concern about high pore pressures on the mylonite seams and the higher coal and shale strata which outcrop in the dam foundation and on the bed of the reservoir. The permeabilities of the upper shale and coal layers were measured in drillholes to be between 10^{-3} and 10^{-4} cm/sec. The effect on the stability of the dam of high pore pressures on the mylonite seam was examined by a special committee including Dr Ralph Peck. It was found that an adequate factor of safety existed on the mylonite seam at a dip of 5° downstream as long as uplift pressures did not rise above elevation 1,850-ft (542 m), the level of the surface of the rock forming the right abutment bench.

Seepage through and drainage from the dam was controlled by its passage through zones of progressively increasing permeabilities, as follows:

Zone	Name	Permeability (cm/sec)
1	Core	10^{-5} to 10^{-6}
2	Transition	10^{-4}
3	Filter	10^{-3}
4	Drain	10
6	Shell	10^{-4}

The Drain was both a foundation blanket and a near vertical ‘chimney’ (wall) drain to collect all seepage and not allow it to pass into the downstream Shell fill. The Drain was separated from the Shell by Filter above the blanket and downstream of the chimney.

[11] Taylor H & Chow Y M (1976) Design, monitoring and maintaining drainage system of a high earthfill dam. ICOLD 12th Congress Mexico Q45 R10 pp147-167.

C49 FOUNDATION PERMEABILITY AND GROUTING

Permeability tests in the rock foundation found that rock formations within 12 m (40-ft) of foundation level had an average permeability of 10^{-3} cm/sec. Rock from 12 m to 33.5 m (110-ft) deep had a permeability of about 10^{-4} cm/sec. Deeper rock was less pervious (10^{-5} cm/sec). The effects of high hydrostatic pressures and high seepage flows were examined by various studies. It was estimated that without a grout curtain, pore pressures would be high and seepage through the dam foundations would be about 1,000 litres/sec. Such pressures and seepage flows were not acceptable and it was decided to grout the foundation rock and install a drainage system downstream of the grout curtain. An early finite element anisotropic seepage model was developed to examine the effects of various grout curtain and drainage configurations on seepage and uplift pressures. The grout curtain was designed to reduce the permeability of the upper rock to about that of the rock at depth. A multi-line (usually five-line) curtain to 107 m (350-ft) reduced permeability of the upper rock by 100-times and by 20-times at depth, to a permeability of at least 10^{-5} cm/sec. This was effective in reducing uplift pressures by as much as 70% from the ungrouted situation.

C50 DRAINAGE TUNNELS

Drainage tunnels (7-ft by 8-ft) were excavated downstream of the grout curtain at about tailrace level and drainage holes drilled upward and downward. The drain hole spacing was 50-ft generally, and 25-ft directly upstream of the underground powerhouse on the left abutment. The spacings were set following analysis, and were wider than usual practice suggested, and there were provisions to drill additional holes if pressures turned out to be higher than expected. This had not proved necessary by the date of the paper (1976, or after as far as is known). Pressure at one deep piezometer close to the bottom of the grout curtain rose to about 70% of full reservoir head, but piezometers in the foundation downstream increased only slightly (about 10% of reservoir head) and remained steady. Foundation drainage into the drainage tunnels is collected in gutters in the tunnels and measured in Parshall flumes. It is discharged at the tunnel portals or into the unused diversion tunnels.

C51 DAM SEEPAGE MEASUREMENT ZONES

Seepage from the embankment dam was directed to seepage measurement chambers. The foundation is divided into four zones (right flank, right terrace, canyon, left flank) by ‘splitter walls’ constructed locally in Shell material through the base of the Transition, Filter and the near vertical chimney Drain, and the blanket Drain on the foundation. Records to 1974 showed total measured seepage of about 17 litres/sec (0.6 cfs) mostly collected at T6 in the tunnel upstream of the powerhouse (with the closely spaced drainholes at 7.6 m centres). The right abutment seepage was about 26 l/s (0.9 cfs) most from the long right abutment of the embankment. The right abutment drainage tunnels collected 9 l/s (0.3 cfs). No seepage was collected (to 1974) in the canyon seepage measurement chamber (R1), but this was thought to be because seepage was bypassing the chamber and discharging through fractured rock near the tailrace tunnels which pass through the area into the tailrace channel. (Seepage first appeared in R1 in 1984).

Taylor & Chow (1976) report that the seepage measurement weirs and piezometers were well maintained. The drain holes, gutters and flumes were inspected monthly. All the instruments generally functioned satisfactorily in the years to 1974 but Measuring Stations 2 and 3 (in the right abutment) were obliterated by a slide of spoil material soon after the reservoir started to fill and were not restored for two summers. The piezometers in the former river bottom gave false readings because they collected methane gas discharged under pressure from the coal seams.

Appendix D: Permeability and Condition of Foundations

APPENDIX D

PERMEABILITY AND CONDITION OF FOUNDATIONS

CONTENTS

LIST OF FIGURES

APPENDIX D

PERMEABILITY AND CONDITION OF FOUNDATIONS

D1 INTRODUCTION

Three issues for consideration in explaining the behaviour of Bennett Dam which relate to the foundation are:

(1) The effects of the foundation on the seepage flow patterns and how these have varied with time since first filling.
(2) Whether there has been erosion or deterioration of the grout blanket and curtain, and other foundation treatment which could explain these variations.
(3) Whether there has been erosion of the embankment materials into open defects in the foundation of the dam.

This section of the report discusses these issues. The discussion centres on the conditions at Instrument Planes 1 and 2. Figure D1 shows the locations of foundation piezometers.

D2 DATA SOURCES

The data for this assessment is taken mostly from the following documents:

(a) Report No H1756, Design Report.
(b) Report No H1973, Report on 1987 Investigations.
(c) Drawings 1006-C02-D972 to D980.
(d) Plots of piezometer readings.

D3 GEOTECHNICAL MODELS AT INSTRUMENT PLANES 1 AND 2

D3.1 Instrument Plane 1

(a) Stratigraphy and structure.

Figure D2 reproduced from drawing 1006-C02-D978 shows the stratigraphy, foundation treatment and instrumentation at Section F, Instrument Plane 1. Figure D3 taken from Figure 5-3 in Report MEP400 also shows the stratigraphy and gives a better detail of the grout curtain and the blanket grouting. Note that the stratigraphy in Figure D2 better matches the drill logs for DH87-10 and has piezometer P1A over the contact between the N6 shale and N6 sandstone.

Figure D4 shows conditions at Section E, quite close to Section F, and is taken from drawing 1006-C02-D977. Figure D5 shows the detailed conditions for DH 87-10 taken from Figure 4-8 of Report H1973.

From these and the information in the reports it can be seen that the foundation consists of interbedded shale and sandstone with minor coal which dips downstream at between 5 and 10 degrees.

The Design report indicates that the sandstones are sparsely jointed. Steeply dipping joints are present, one set striking northwest, the other northeast. A weaker set strikes north.

(b) Rock Mass Permeability

The Design Report H 1756 Section 3.3.4 indicates that the thick sandstone and shale beds are impermeable. Groundwater flow is mostly along top bedding planes of the shale layers, and within mixed shale and coal layer. It indicates that the upper 40 ft weathered surface had an average permeability of 10^{-4}cm/sec and the rock at greater depths a permeability of 10^{-5} cm/sec.

These are consistent with the water pressure test results in BH 87-10 (see Figure D2 for location) which has permeability's generally less than 10^{-5} cm/sec or 1 lugeon except at the contacts between the N6 sandstone and the N6 and N7 shales where the permeability is 2 to 8 x 10^{-5} cm/sec, or 2 to 8 lugeons.

However there is considerably more data on the permeability of the foundation from the water pressure testing carried out as part of the grouting program during the dam construction. The curtain grouting in the canyon section was carried out from the grouting gallery. The curtain grouting was preceded by blanket grouting over the areas shown in Figures D2 and D3.

Water pressure testing was carried out on some grout holes. Those tested were at about 150 ft spacing. Each hole was tested over 15ft sections. The outer two lines of grout holes (Lines 2 and 4) were drilled at an inclination of 30 degrees to the vertical along the axis of the dam, and about 12 degrees in the upstream / downstream direction so would have intercepted sub vertical defects. Assuming that the holes tested were drilled as primary holes, there would be little effect on the water pressure testing by grouting from adjacent primary holes so the tests can be regarded as representative of the in-situ conditions.

Grouting of these primary holes was systematically followed by secondary and tertiary grout holes so the minimum grout hole spacing in the horizontal direction was 37.5 ft. or 11 metres. Closure was controlled by grout takes, and quaternary holes were drilled and grouted in areas where tertiary holes had significant grout takes.

The primary and secondary grout holes penetrated to RL 1450ft; that is through the N6 shale and about 30 ft into the N7 sandstone below. Tertiary grout holes were drilled to higher RLs.

Grout pressures were quite high. For example 280 psi (approx 2MPa) at 25ft to 50 ft below the gallery floor, or as close as 40ft from the foundation surface.

Grout takes were high in some strata; in particular around the contact between the N5 shale and the N6 sandstone with takes of up to 750 cubic feet of solids over a 20 ft length. Assuming that the solids were dry cement this represents > 3000 kg/m of cement. High grout takes coincided with the higher lugeon values but the high pressures may have induced hydraulic fracture in some areas.

Grout takes in secondary holes were lower than primary and tertiary takes were generally, but not always, low.

These data have been marked up on sections to show the pattern of lugeon values and grout takes.

The central line of grout holes was drilled vertical with primary and secondary holes to RL about 1380 ft. Tertiary holes were drilled generally to about RL 1510ft, into the N6 shale but not through it.

Lugeon values and grout takes are much lower than for the outer lines indicating grout was penetrating between holes quite effectively.

(c) Stress Relief Features.

Section 3.3 (c) of the Design Report indicates that in the river bed, vertical stress relief has resulted in the formation of fractures in the N5 sandstone that extended as far as 41 ft below the surface. These features were infilled with sand and or silt and rock fragments. They "pinch and swell markedly within short distances" The bedding plane sand seams were up to 14 inches (350mm) wide. These seams are described further in Section 4.1.3 of Report H1973.

Figure D6 which is taken from Figure 4-2 in Report H1973 shows the extent of the foundation affected by stress relief. In the vicinity of Instrument Plane 1 the stress relief effects generally extend up to 30 ft depth, and locally up to 60 ft.

It is noted that the alluvium infilling of the open stress relief features does not extend more than about 180 ft downstream of the grout gallery centreline and only beneath about the upstream 60% of the core. They do not extend beneath the transition or filter.

(d) Foundation Blanket Grouting.

The area beneath the core was blanket grouted. The blanket grout holes were vertical, 20 ft deep at spacing 10 to 17ft in most of the valley section. In some areas towards the downstream side of the core / transition contact the holes were generally 15ft deep. The 1987 investigations involved drilling several boreholes through the grouted blanket with water pressure testing to determine the permeability in sections of the boreholes. These showed that the foundation treated by blanket grouting generally had low permeability. The results are shown in Figure D7 taken from Figure 4-6 of Report H1973. The permeabilities were all below 5.2 E-07 m/sec, or 4 lugeons, except for one test near the downstream end of DH 86-3 which was 6.5 E-06 m/sec, or 50 lugeons. Caving prevented water pressure testing of some holes. The estimated average vertical permeability of the blanket grouted rock from Report H 1973 was 5 E-08 m/sec.

The stress relief sand seams were treated by washing some of the infill between grout holes and then grouting the open passages. This was done for a width of 60 ft either side of the grouting culvert and to a depth of about 60 ft. This extent is shown best in Figure D3 and Figures D5 and D6. Lines of washing and grouting were constructed further downstream as shown in Figure D6.

The investigations carried out in 1987 showed that the blanket grouted N5 sandstone had a low vertical permeability. It is likely that the horizontal permeability could be significantly greater due to the presence of the sand filled stress relief features.

From these data the following permeability model can be interpreted:

N5 sandstone after blanket grouting and washing of alluvium in stress relief defects	3 lugeon vertical 30 lugeon horizonta
N5 sandstone after blanket grouting and limited washing of alluvium	5 lugeon (??, limited data)
N5 sandstone no grouting outside stress relief area	At least 5 lugeon (limited data)
N6 shale and 10 ft into the N6 sandstone,	25 lugeon
N6 sandstone excluding top and bottom 10ft	0.1 lugeon
Bottom 10 ft of N6 sandstone and top 20 ft of N7 shale	15 lugeon
Central 10 ft to 15 ft of N7 shale	0.5 lugeon
Lower 10ft of N7 shale and top 20 ft of N7 sandstone	10 lugeon
Below top 20 ft of N7 sandstone to RL 1380 ft at least	0.5 lugeon

The grouting appears to have been effective in reducing the permeability in the N6 shale and the N6 shale N6 sandstone interface to about 2 lugeon with a likely effective width of 30 ft.

The permeable strata below the N7 shale may not have been as well grouted because the central vertical line does not penetrate that deep. The grout takes in the outer holes do reduce from the primary takes. It may be 5 lugeon with an effective width of 30 ft, but more permeable effectively non-grouted sections may exist. This seems to be the case because two piezometers in the N7 shale show near reservoir level pressures indicating the grout curtain is ineffective.

D3.2 Instrument Plane 2

(a) Stratigraphy and structure.

Figure D8 reproduced from drawing 1006-C02-D975 shows the stratigraphy, foundation treatment and instrumentation at Section C which is at Instrument Plane 2. Figures D9 and D10 taken from drawings 1006-C02-D973 and D974 show stratigraphy and foundation treatment at Sections A and B.

Figure D11 reproduces Figure 5-4 from Report MEP 400. It shows the piezometric heads in the embankment at IP 2 in 1997. Note that the geometry of the core in this figure is incorrect. Figure D9 shows the actual zoning.

From these the core of the embankment at Instrument Plane 2 is founded on the N3 sandstone.

(b) Rock Mass Permeability.

The best available data is from the grouting records for Section VII, Stn 20 to 25 approx. This is probably further towards the right abutment but is on the same wide plateau as IP2 so is probably a guide.

There are 5 lines of grouting, the outer lines 1 and 5 are vertical and only about 45 ft deep. Lines 2 and 4 are inclined at 30 and 40 degrees to the horizontal in opposing directions! These penetrate to about 15 ft below the N3 shale. The central line 3 is vertical and penetrates to about 15ft below the N4 shale.

The primary holes are spaced at 120 ft centres, with secondary and tertiary holes drilled and grouted for the outer four lines, with quaternary holes as required.

From these data it is assessed that a reasonable model of the permeability is:

Upper 40 ft of N3 sandstone	2 lugeon
40 ft to 60 ft in N3 sandstone	40 lugeon
From 60 ft to within 10 ft of the base of the N4 upper sandstone including the N4 upper shale	2 lugeon
From 10 ft above the base of the N4 upper sandstone to the top of the N4 lower shale	0.5 lugeon
N4 lower shale	35 lugeon
N5 sandstone	1 lugeon

The high permeability at 40 to 60 ft seems likely to be a stress relief feature.
The grouting seems to have closed down to about 0.5 lugeons with an effective width about 30 ft.

The Grout Blanket Construction Report by IPEC has drawings showing the expected permeability of the blanket grout based on the closure achieved. These are between 1 and 3 lugeons, so it would be reasonable to adopt 3 lugeons as the permeability of the blanket.

D3.3 Benchmark No 1

It is not possible to tell accurately where Benchmark No 1 is in relation to the grout station system, but in any case the grouting data for that area is not available.

In the absence of data it seems that the conditions are likely to be similar to those for Instrument Plane 2.

D3.4 Benchmark No 2

The grouting data indicates moderate lugeon values and grout takes down to RL 1800. These are not related to stratigraphy and are almost certainly related to valley stress relief.

A reasonable estimate of the permeability of the rock strata from the surface to RL 1800 (rock units N2 sandstone, N2 shale, N3 upper and lower sandstone, N3 upper shale) is 20 lugeons. Below that it is about 5 lugeons.

Only the upper 60 ft is intensively grouted. Below that grout holes are very widely spaced. The effective grouted permeability in the upper 60 ft is probably about 2 lugeons with an effective width of 30 ft.

D4 OBSERVED PIEZOMETRIC PRESSURES IN THE FOUNDATION

D4.1 Instrument Plane 1

Figures D1 and D2 show observed piezometric levels in 1997. It can be seen from these and piezometer plots that:

(a) There are high pressures in piezometers P1A in DH87-10, FP43, and FP41B. These piezometers are in or on the margins of the N7 shale unit. The piezometers fluctuate with reservoir level. There is not a great drop in pressure through the grout curtain. Piezometer FP33B at the toe of the dam also shows high pressures in this rock unit.
(b) Piezometer P3/P3A in DH 87-10 has pressures lower than the piezometers discussed in (a). These also vary with reservoir level. The pressures gradually increase with time, but there was a step in readings in 1997 due to a change in the piezometer installation and the zone being measured, and erratic readings 1997 to 2002. This piezometer is on the contact between the N6 shale and the N6 sandstone. Piezometers P4 /P4A in DH 87-10 had similar pressures from 1992 to 1997, but lower pressures after the installation was changed.
(c) Piezometers FP40, FP 44, 45 and 46 all have pressures at about tailwater level. They are in the N5 and N6 shale rock units.

(d) Piezometers P3, P4 and P6 in DH 86-1 are located in the grout blanket. They show pressures which are highest in the upstream piezometer and lowest in the downstream. The readings after 1997 when the tips were located in a short section of borehole, are different to the earlier readings, and probably not as useful as they may be isolated from the groundwater. Piezometers P2 and P1 in DH 86-1 are located below the grouted zone and show low pressures but above tailwater level. The horizontal gradient within the grouted blanket varies from about 1.5H:1V from P6 to P4, 1H:1V P4 to P3, and 0.25H to 1V P3 to P1.
(e) The pressures in the embankment piezometers D1, D2, D3 and D4 just above the blanket grouting also are lower at the downstream than upstream, and are higher than the piezometers in the blanket.
(f) The following piezometers have shown trend changes with time:

FP 40, dropped about 20 ft from 1972 to 1987, relatively constant since then after some irregularity in 1998 to 1992.

FP 41A, dropped 150 ft approx from 1968 to 1987 and has been constant at tailwater level since 1987.

FP41B, almost at reservoir level up to 1982, then peaks were lower than reservoir peaks, with another reduction from 1999.

FP 43, pressures have reduced with time from a peak of 2160 ft approx, to around 2050 ft in 1984, and to 2000ft in 2011; irregular readings from 1987 to 1992.

FP 47, increased gradually from 1970 to 1997.

P3 in DH 87-10, gradually increases with time.

P4 in DH 87-10, increased with time from 1990 to 1992/3.

From the information discussed above it can be concluded that:

(a) There are relatively low pressures in the N5 sandstone below the blanket grout. These are lower than the embankment pressures and lower than the pressures in the contact zones discussed in (a). The pressures reduce with distance downstream of the grout culvert. It is likely that this is a result of relatively high horizontal permeability in the N5 sandstone which is affected by stress relief and the presence of sand seams. This stratum is acting as a drain to the higher pressures above, below and upstream.
(b) The pressures in the N5 shale, N5 sandstone and N6 shale downstream of the centreline are at tailwater level.
(c) There are high pressures within the zone of sand treatment and grouting to 60ft downstream of the grouting culvert. This seems to indicate that grouting was successful as the pressures are contained. There are some gaps noted between the concrete of the culvert and the surrounding rock.
(d) The pressures in piezometers FP40, 41A, and 43 reduced significantly from 1972 to 1987. This coincides with the major drop in pressures in embankment piezometers D1 to D4 and this is the likely reason for the drops. It is almost a certainty that the underdrain effect of the stress relief zone in the N5 sandstone is affecting the seepage flow nets and pore pressures in the dam and in the foundations.
(e) The pressures in the other horizontal boreholes drilled through the grouted blanket in 1987 are much higher than in DH86-1. From this it appears that the underdrain effect of the stress relief features in the N5 sandstone is less in these areas. This is consistent with the extent of stress relief features mapped in Figure 4-2 of Report H1973. It means that Instrument Plane 1 may not be typical of pore pressures in the foundation and,

because the embankment and foundation act together, of the embankment pore pressures.

D4.1 Instrument Plane 2, and Sections A and B

Figures D7, D8 and D9 show observed piezometric levels in the foundation and the embankment in 1997. It can be seen from these and piezometer plots that:

(a) The piezometric pressures in the upper foundation strata are low on this section and Sections A and B. This may be due to the grout curtain and / or the drainage system installed from the drainage tunnel being effective.
(b) Piezometer FP 15B has high pressures. It is located in the N5 shale but possibly overlaps the N5 sandstone / N6 shale contact. The pressure in FP 15A is lower indicating the lower strata are confined. The pressures vary with the reservoir level.
(c) Piezometer FP 16A shows a higher pressure than FP16b and is possibly perched.
(d) The pressures in piezometers EP 60 to 63 are higher relative to the total head above the foundation than in Instrument Plane 1. They decrease somewhat from upstream to downstream.
(e) There are no piezometers which provide data on the effectiveness of the blanket grouting.
(f) The following piezometers have shown trend changes with time:

FP 15A shows very little variation with time. There was a 10ft approx. rise from 1969 to 1983.

FP 15B peaked in 1972 about 75 ft higher than present readings. It reached essentially steady conditions in 1978.

EP 60, 61 62 and 63 all reached a peak in 1973-74, and a second peak in 1981-82. For EP 60 and 61 the second peak was the highest. The first of these peaks is seen in F15B but the second is not.

From the information discussed above it can be concluded that:

(a) There is much less data on this Instrument Plane than for Instrument Plane 1, so the conditions are not so well known.
(b) It is likely that the 40 lugeon strata from 40 to 60 ft in the N3 sandstone and the underlying strata are acting as a drain to the embankment. This is contrary to the observed piezometric pressures in the embankment being relatively higher than in Instrument Plane 1.

D5 FOUNDATION TREATMENT BENEATH THE CORE, TRANSITION AND FILTER

D5.1 Data Available

Section 7.3.2 of the Design Report indicates that the rock surface beneath the core was cleaned with air / water jets, the shale beds covered with 2 to 3 inch layer of gunite, and areas of vertical jointing in sandstone were sealed with slush grout. In many places weak and fractured rock was rock bolted prior to guniting.

There is detailed geological mapping and plans showing the treatment carried out of the foundation. This is presented in 100 ft x 100 ft sections. The numbering system for part of these is shown in Figure D6. These cover the Core contact and to a lesser detail the foundation for the Transition (Zone 2) and Filter (Zone 3).

Figures D12 and D13 give examples. The quality and detail of this data is outstanding.

D5.2 Mapping and treatment for core foundation

The whole of the core foundation was mapped in detail.

The foundation for the core was treated by:

- Slush grouting as required to seal any vertical cracks.
- Application of gunite (also called pneumatically applied mortar, or PAM). This was extensive and only areas of unjointed sandstone appear to have not been treated. The drawings say this is to be 2 inches (50mm) thick unless otherwise stated.
- Smoothing of steps in the foundation by concrete.
- Anchoring of concrete as required withstanding uplift from grouting.
- A special program of washing out the alluvium which had filled stress relief joints and bedding partings was carried out for a width of 60 ft upstream and downstream of the grouting culvert centreline. This was to a depth of 60 ft or less if there was no alluvium infill. This did not include beneath the culvert. Details are shown in Figure D6. This treatment was extended to a further four lines downstream as shown in Figure D6. This was not completed as described in that figure as it was considered unnecessary.

From the drawings the majority of the shale areas were covered by gunite.

This standard of mapping and treatment extended over the whole of the Core foundation.

D5.3 Mapping and treatment for transition and filter foundation

The foundations for the Transition and Filter were mapped but not to the same detail as the Core. A check of the mapping for the core in adjacent areas in the canyon area showed that there were no areas of open jointing. This is consistent with Figure D6.

In general gunite treatment did not extend beneath Zones 2 (Transition) and 3 (Filter) stopping at the downstream boundary of Zone 1 (Core).

There is mention of open defects in the foundation of Zones 2 and 3 in area 303 in the upper left abutment but this area was covered in gunite.

It is reasonable to conclude therefore that there were no areas of wide open joints beneath Zones 2 and 3.

The drawings show that steps in the foundation profile for Zones 2 and 3 were treated by concrete.

The drawings of the upper left abutment show that the excavation of weathered rock was carried out for Zones 1, 2 and 3. That is Zones 2 and 3 were founded on the same surface as Zone 1, not on a steep batter slope.

D6 ASSESSMENT OF THE LIKELIHOOD OF EROSION INTO OPEN DEFECTS IN THE FOUNDATION

D6.1 Method of Assessment

Fell et al (2008) suggest that the likelihood of continuing erosion be assessed by comparing the joint opening size (JOS) to the D_{95} of the surrounding soil. The joint opening size which would give continuing erosion is $JOS_{CE} = D_{95}$ surrounding soil. They suggest that D_{95} should be based on the soil grading after re-grading on the 4.75mm particle size.

This is based on application of the Foster and Fell (1999b, 2001) continuing erosion criteria which assume that the crack width is equivalent to the filter opening size of the voids between the particles in a filter. The filter opening size has been shown by Sherard et al (1984) to be D_{15} / 9. This was confirmed by Foster and Fell (1999b).

There is some information to indicate that these criteria may be somewhat conservative. This comes from grouting fractured rock with cement grout which is a suspension of silt sized particles usually with a D95 less than 0.075 mm (Fell et al 1992). A number of authors including Mitchell (1970) and Karol (1984) indicate that grout will not penetrate if the fracture opening is less than 3D100. Littlejohn (1985) indicates that grouting is not possible unless the fracture opening is greater than 5D100. Testing by Tjandraja (1989) showed some grout penetrated for openings of 3D100, but eventually the opening was bridged over by coarser particles.

D6.2 Joint opening Sizes for Continuing Erosion

From Table F8 in Appendix F on the assessment of filter capability, the range of D_{95} of the core after re-grading on the 4.75mm sieve is 0.65mm to 3.8mm.The range of D_{95} of the transition after re-grading on the 4.75mm sieve is 1.9mm to 4.1mm. Fractures as open as these would give lugeon values >> 100 using the data in Fell et al (1992).

The low lugeon values in the horizontal boreholes in the grout blanket in the valley floor are indicative of joints with openings less than 0.1mm based on relationship between lugeon value and defect opening in Fell et al (1992). This means that it is very unlikely that the core would erode through the grouted blanket unless it has deteriorated since 1987. There is no evidence that this has happened.

The lugeon values elsewhere in the core foundation are less than 25 lugeons in the canyon section and 40 lugeons in the upper left abutment. These are indicative of fracture openings of less than about 0.3mm assuming one fracture per metre, and less than 0.5mm for 3 metre spacing defects. The treatment of the foundation beneath the core elsewhere was very extensive and thorough so it is unlikely such defects were not covered by slush grout and gunite.

It can be concluded therefore that there is a very low likelihood of erosion of Zone 1 core into the foundation. If any occurred it would be of very limited extent ("some erosion" in filter terms) because the core would filter against the defects.

Even though the Zone 2 Transition, and Zone 3 Filter foundations were generally not treated with slush concrete or gunite the foundation lugeon values are such that the maximum defect openings would be less than 0.5mm, at most 1mm.

It can be concluded therefore that there is a very low likelihood of erosion of Zone 2 and 3 into the foundation. If any occurred it would be of very limited extent ("some erosion" in filter terms) because Zones 2 and 3 would filter against the defects.

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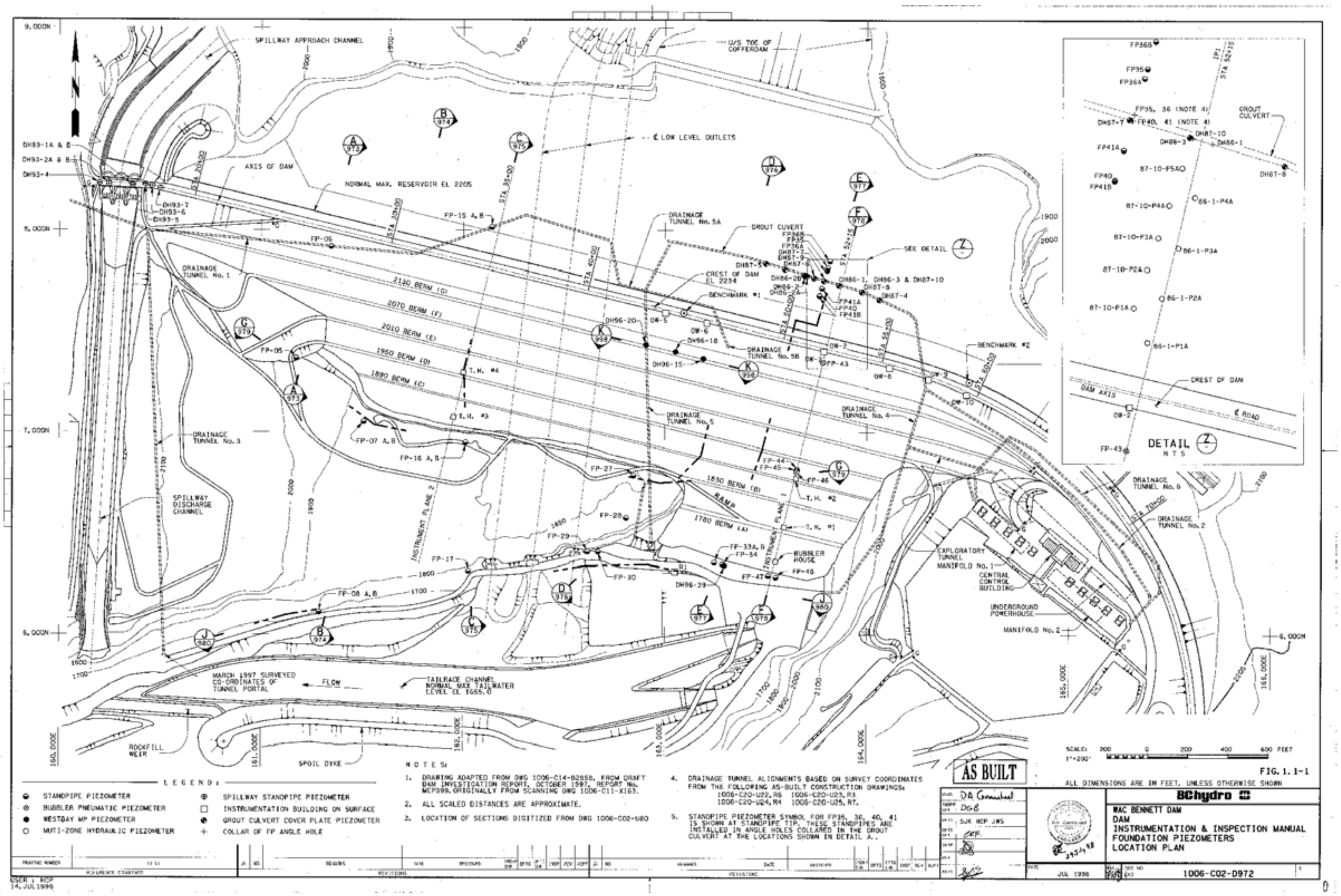

Figure D1: WAC Bennett Dam – Instrumentation & Inspection Manual - Foundation Piezometers - Location Plan

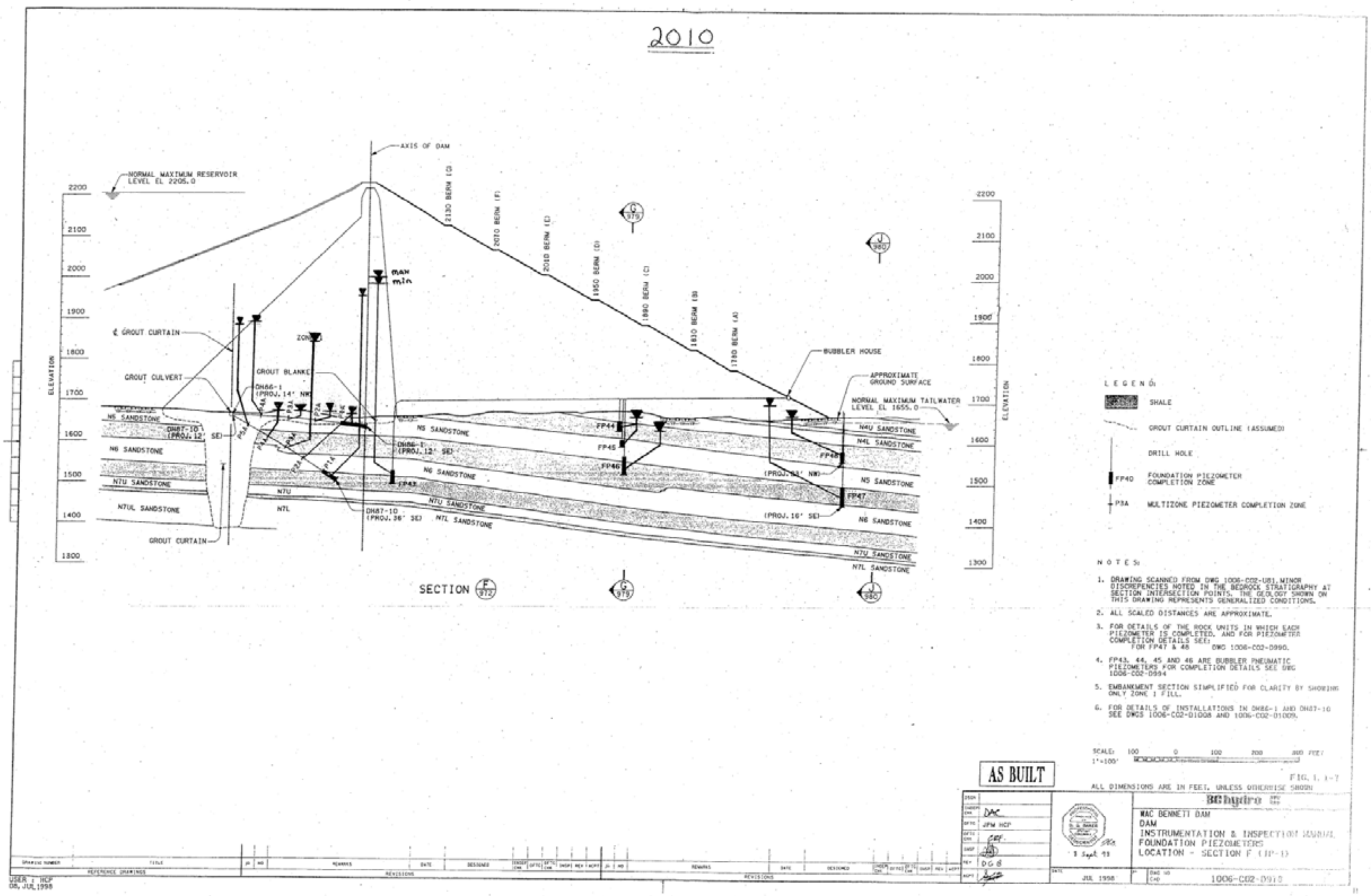

Figure D2: WAC Bennett Dam – Instrumentation & Inspection Manual - Foundation Piezometers – Section F

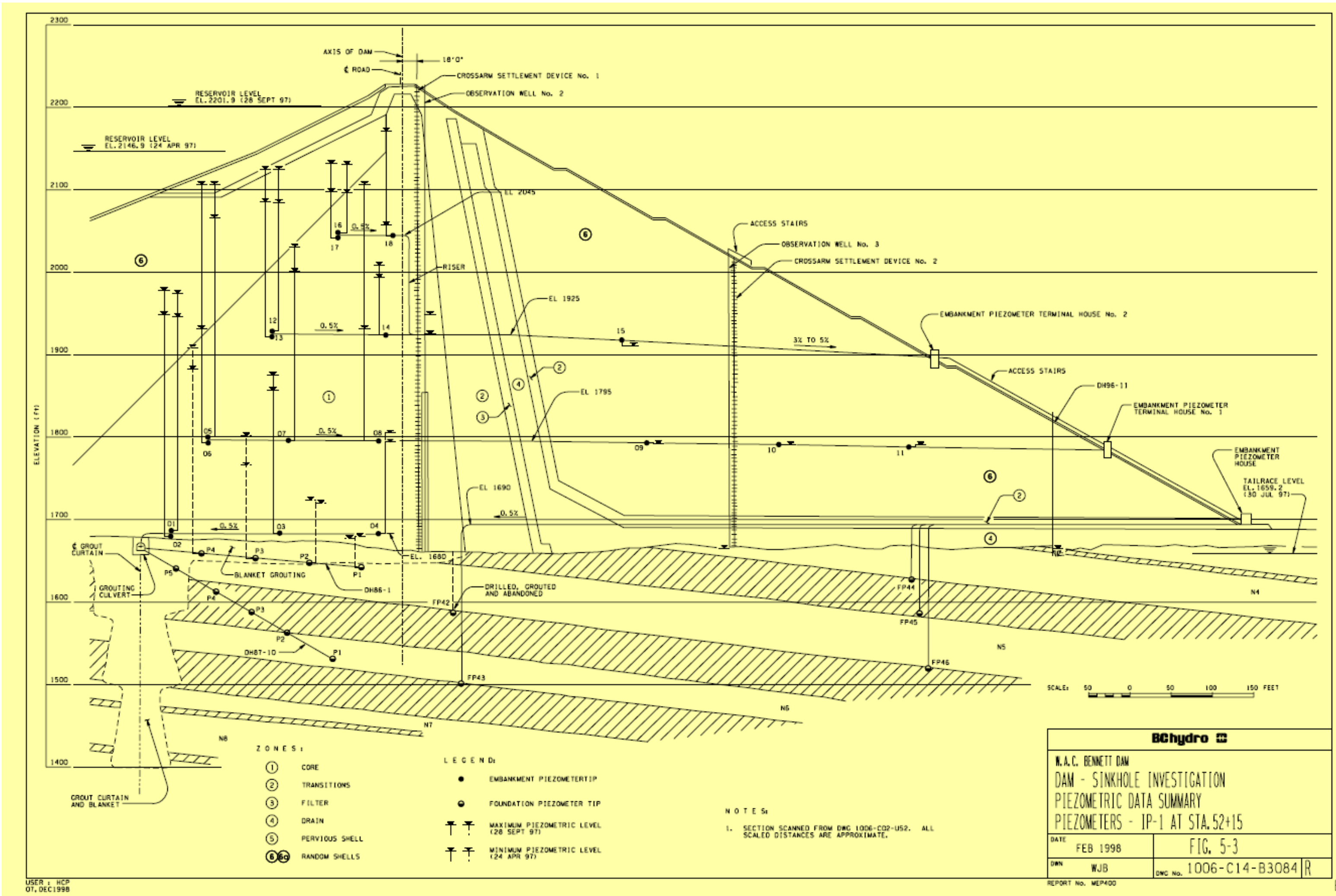

Figure D3: WAC Bennett Dam – Piezometric Summary – Piezometers – IP at STA. 52+15

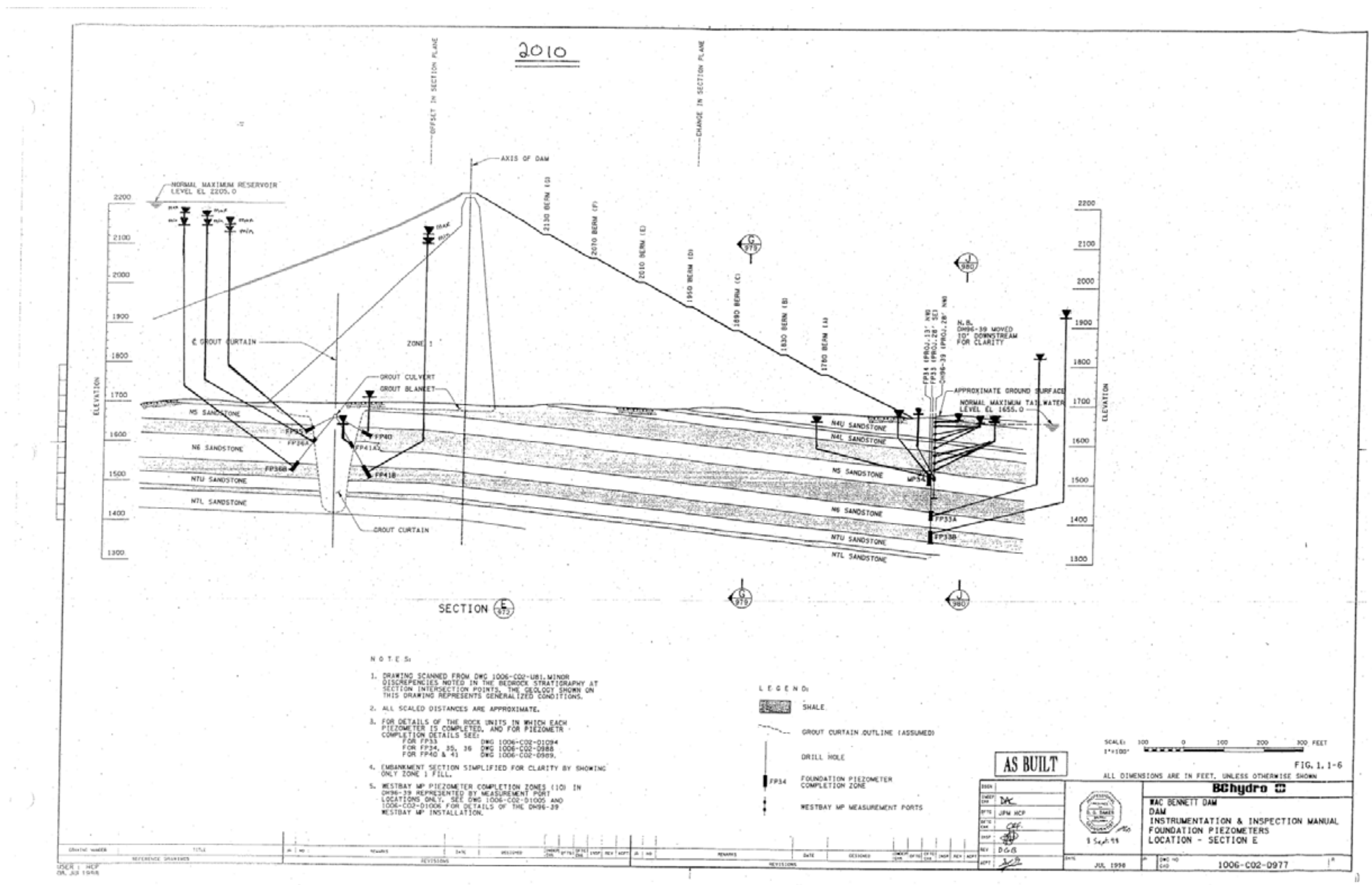

Figure D4: WAC Bennett Dam – Instrumentation & Inspection Manual - Foundation Piezometers – Section E

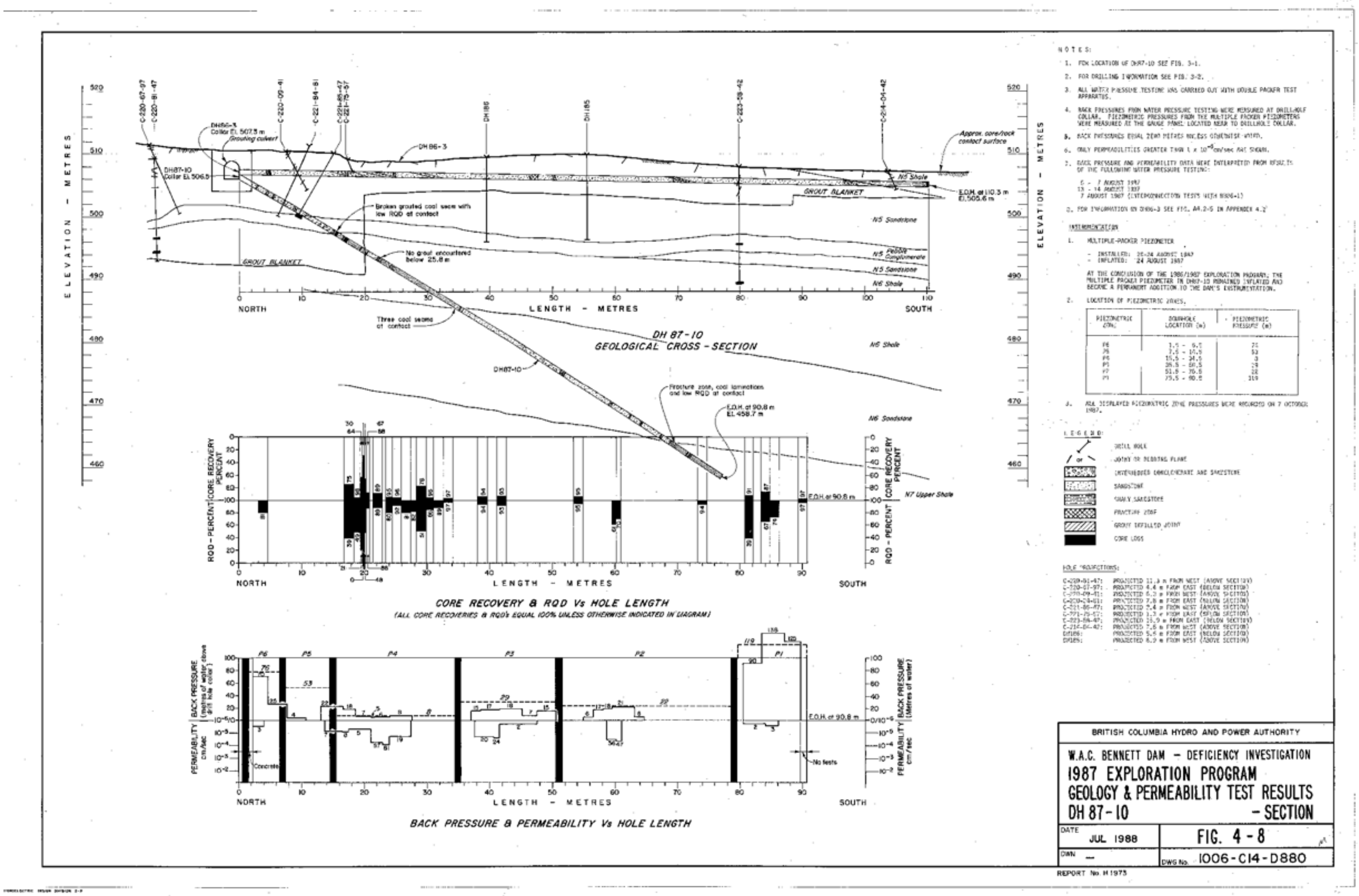

Figure D5: WAC Bennett Dam – Geology & Permeability Test Results Section – DH87-10

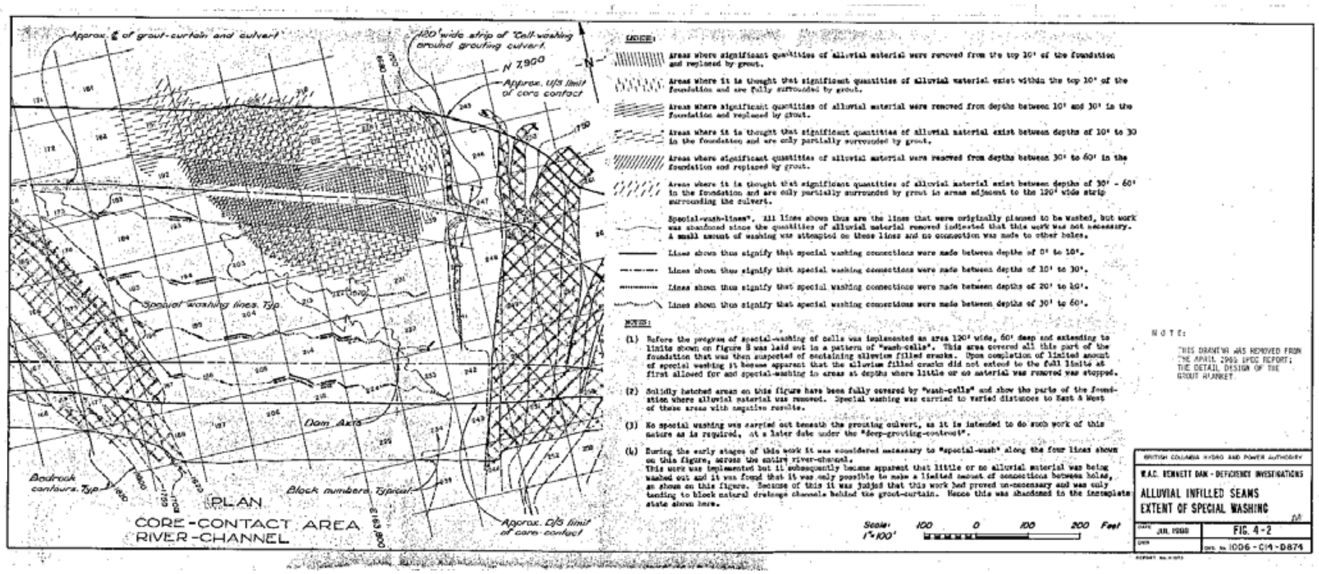

Figure D6: WAC Bennett Dam – Alluvial Infilled Seams – Extent of Special Washing

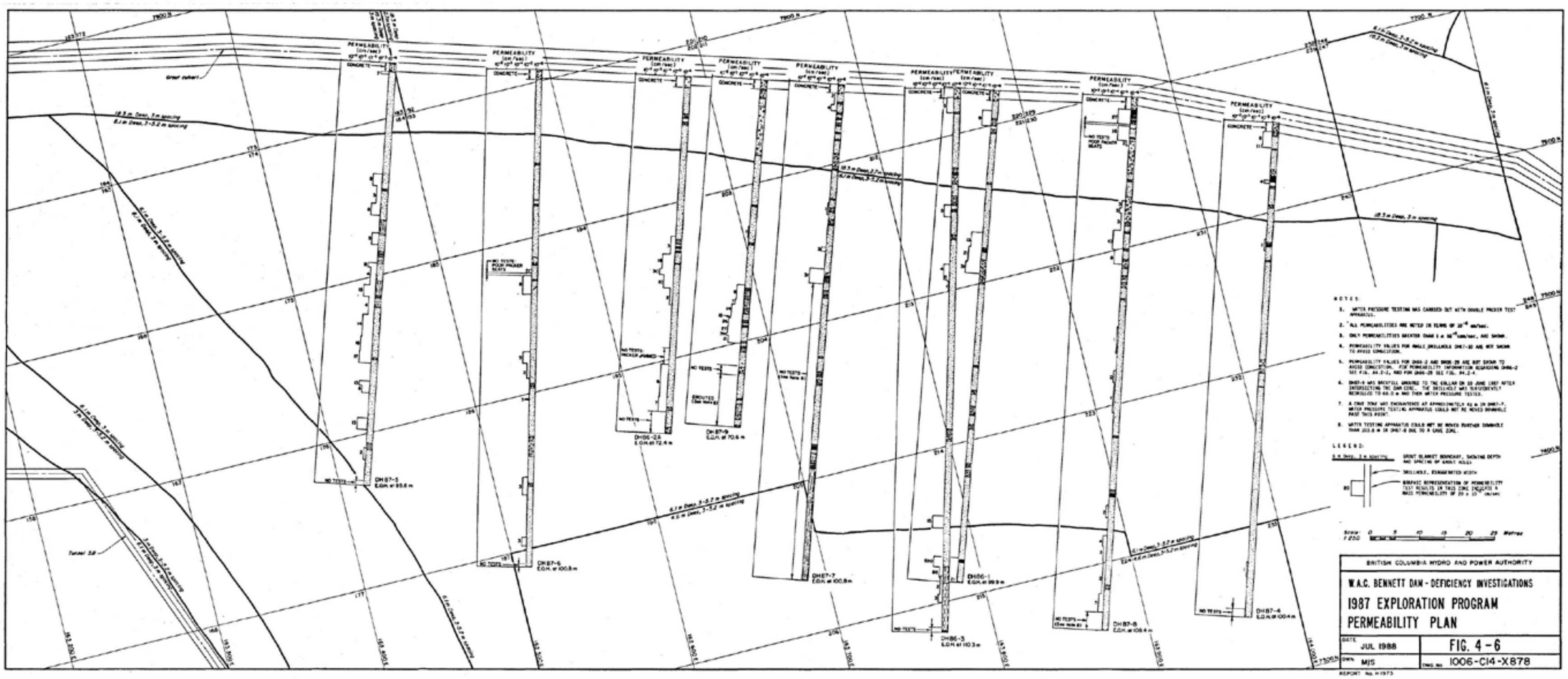

Figure D7: WAC Bennett Dam – 1987 Exploration Program Permeability Plan

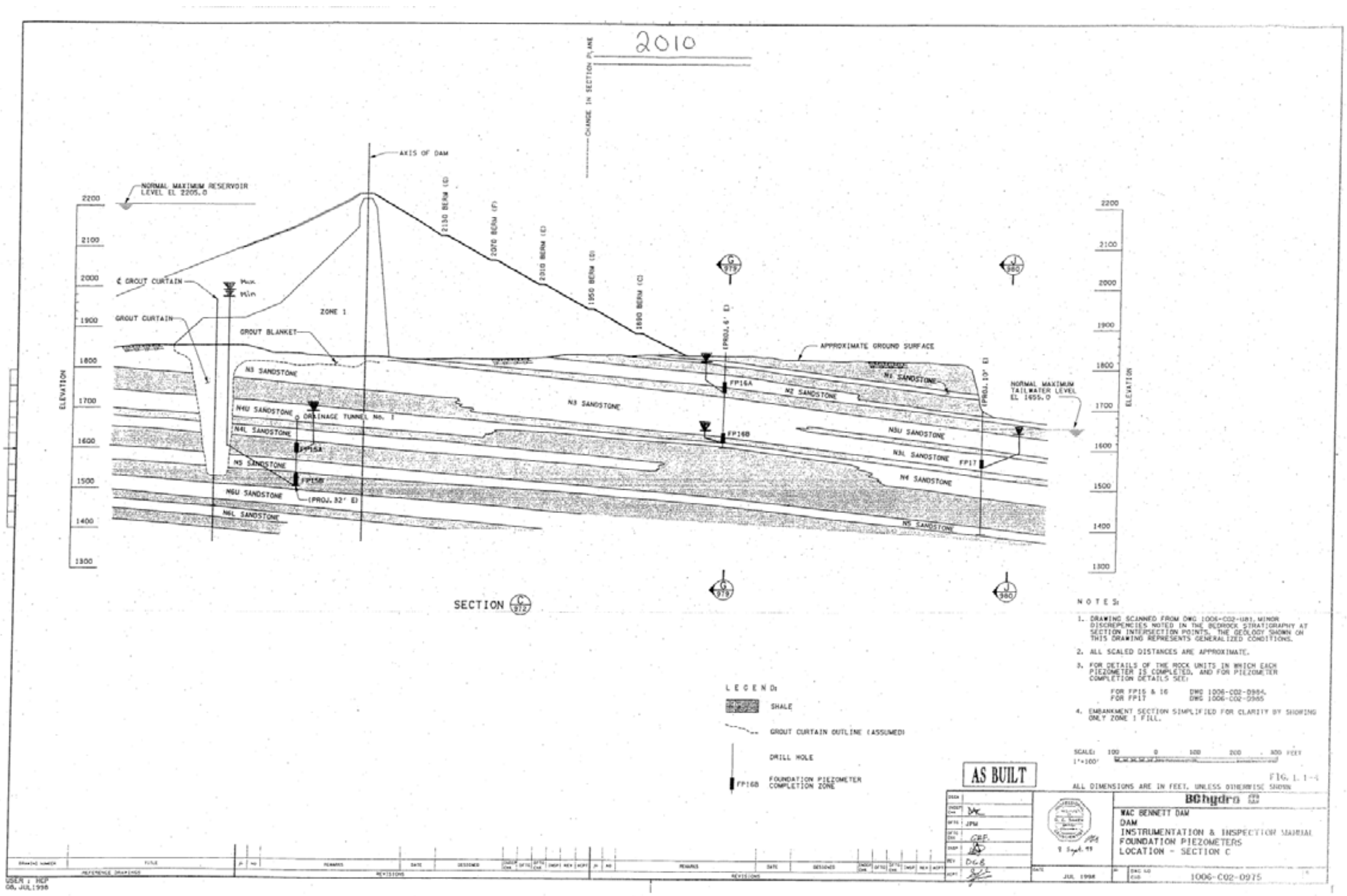

Figure D8: WAC Bennett Dam – Instrumentation & Inspection Manual - Foundation Piezometers – Section C

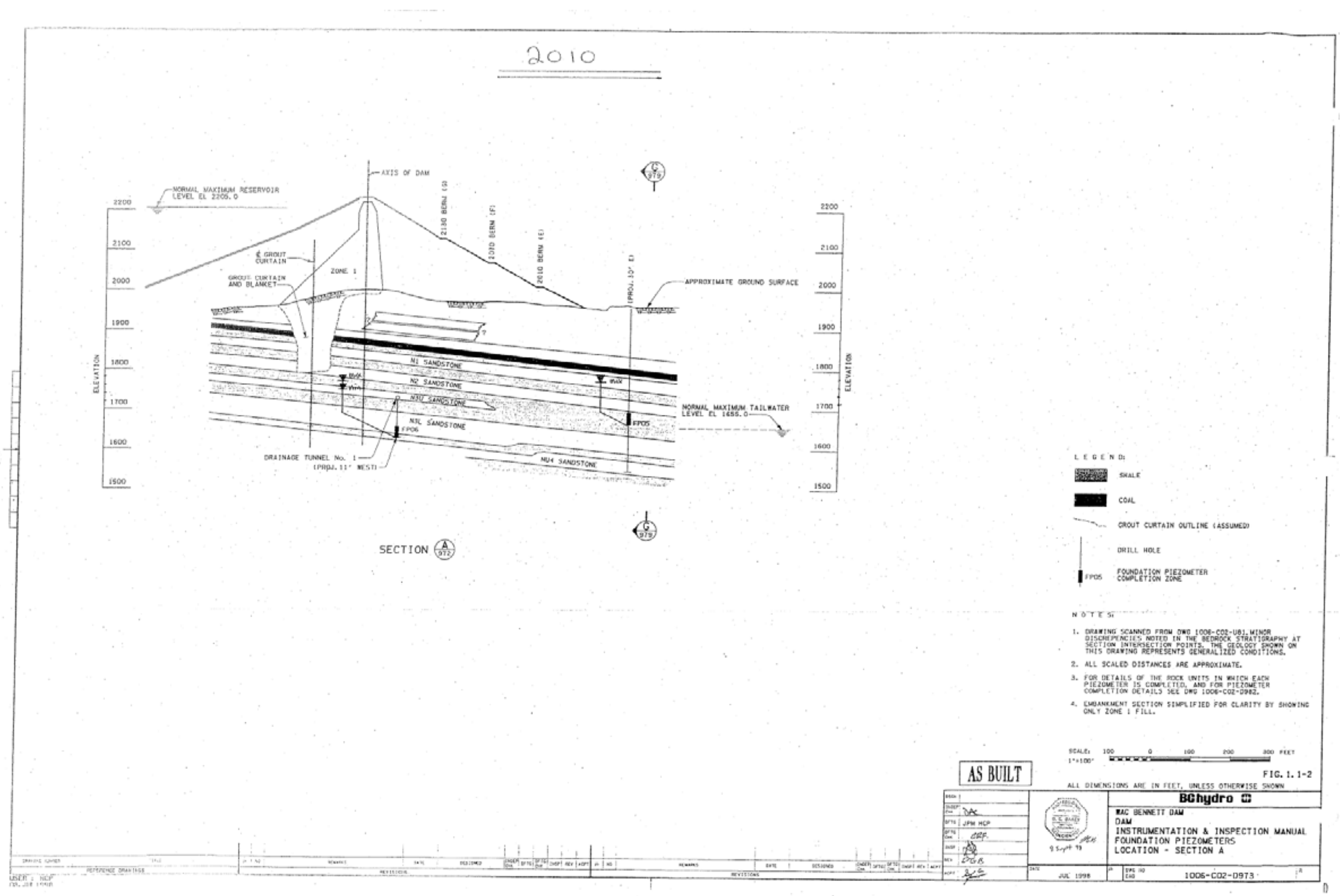

Figure D9: WAC Bennett Dam – Instrumentation & Inspection Manual - Foundation Piezometers – Section A

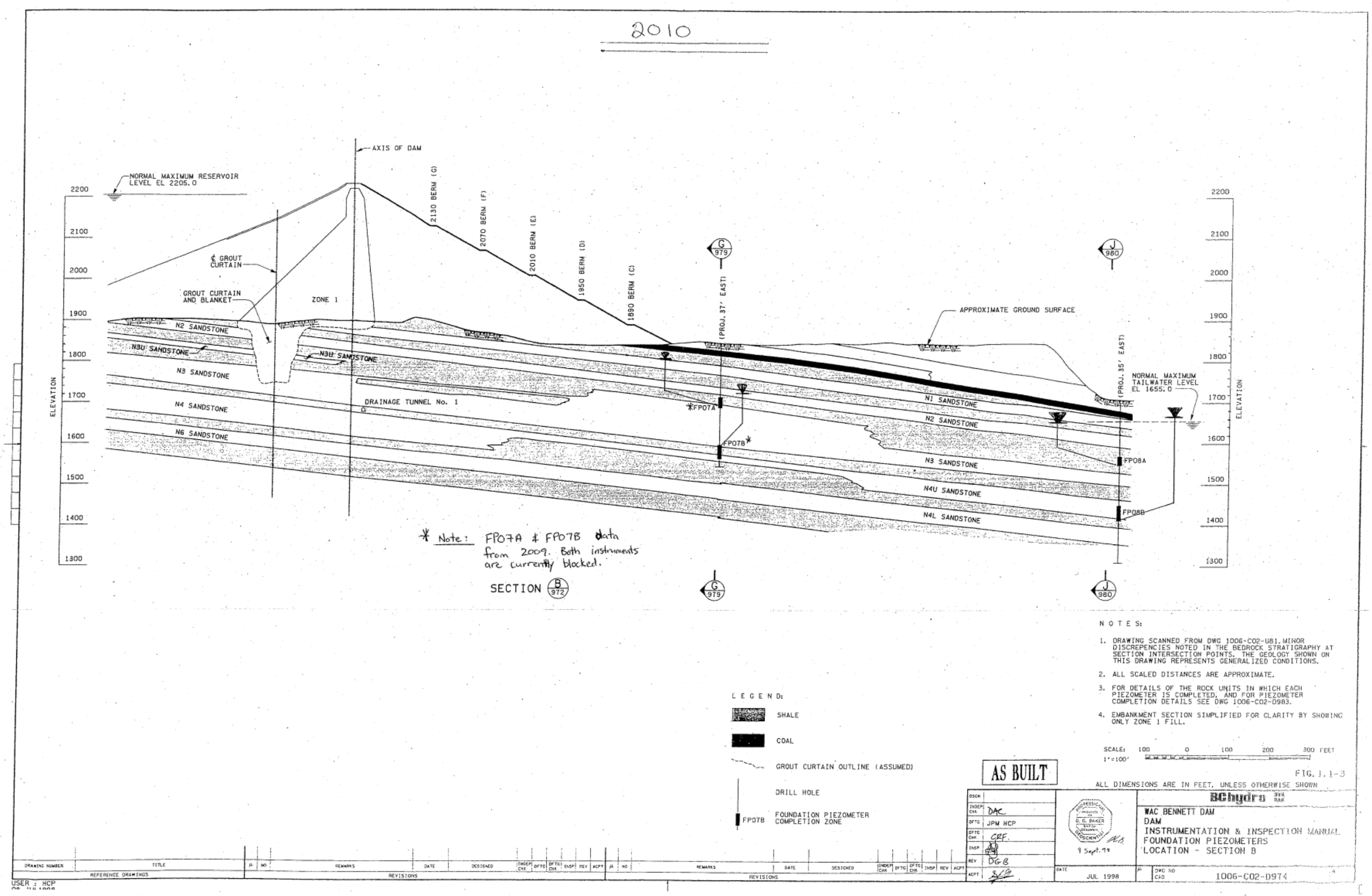

Figure D10: WAC Bennett Dam – Instrumentation & Inspection Manual - Foundation Piezometers – Section B

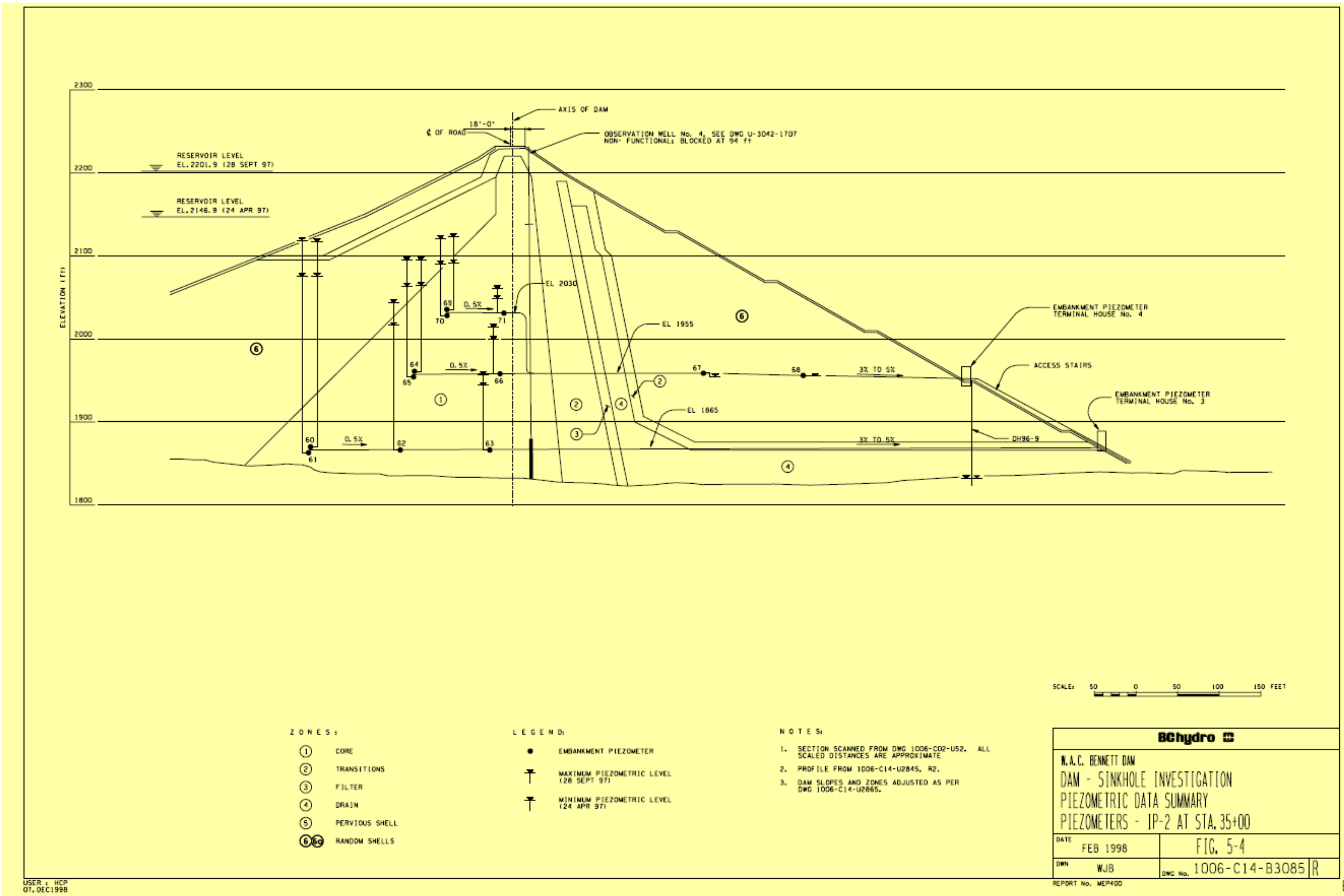

Figure D11: WAC Bennett Dam – Piezometric Data Summary – Piezometers IP-2 at STA. 35+00

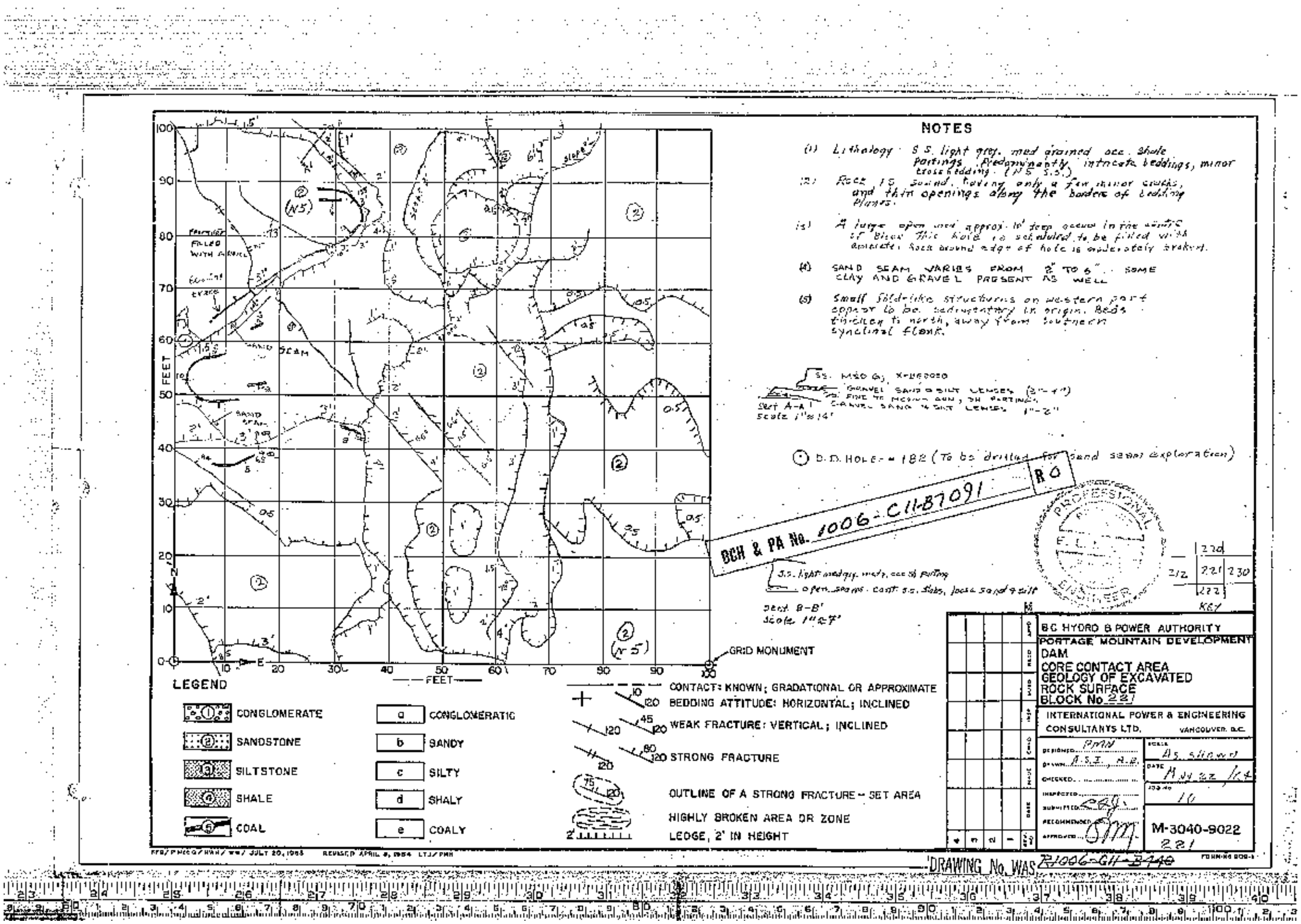

Figure D 12: Example of geological mapping of foundation in core foundation- Area 221

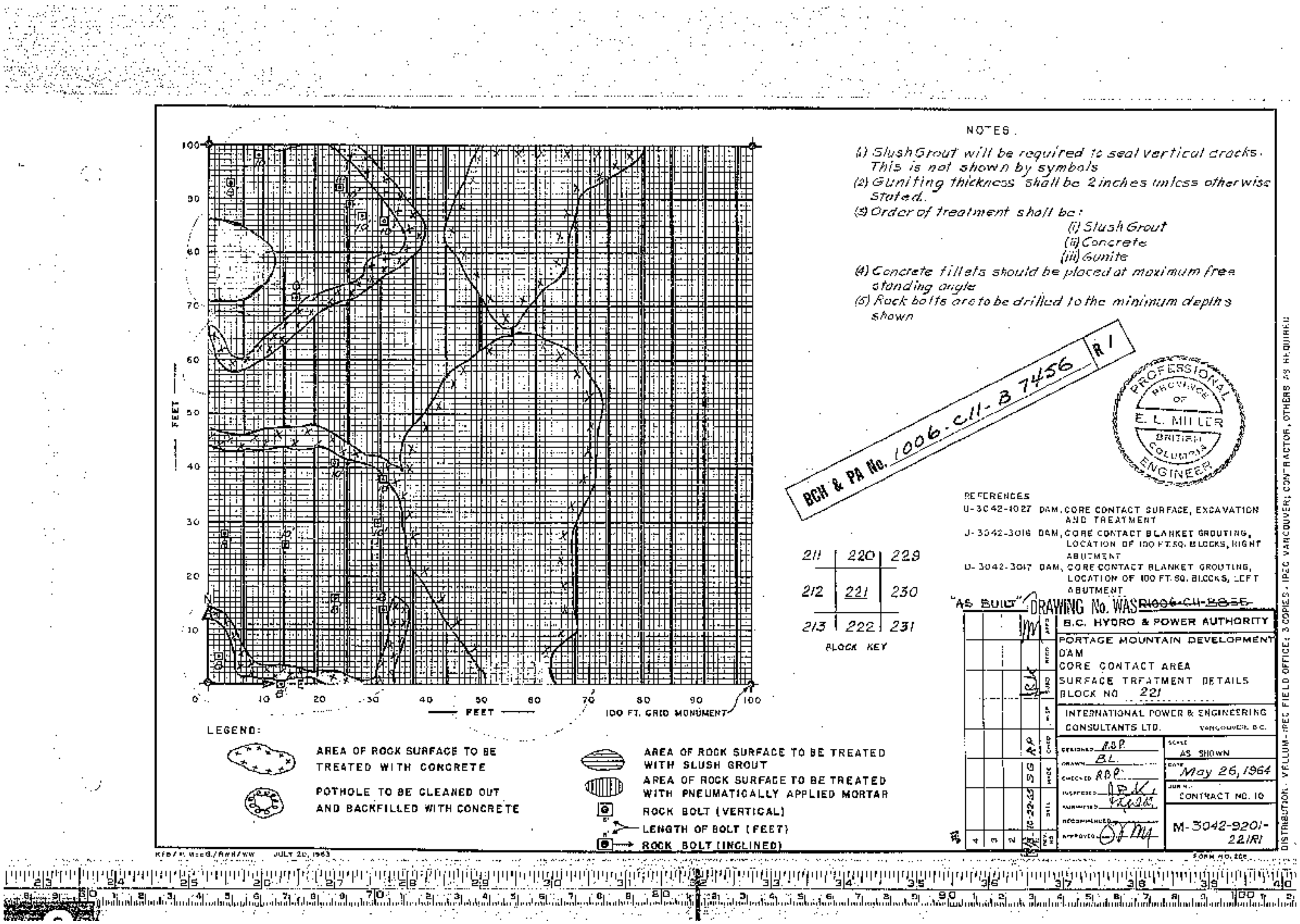

Figure D13: Example of plan of foundation treatment in core foundation

Appendix E: Embankment Permeability

APPENDIX E

EMBANKMENT PERMEABILITY

CONTENTS

LIST OF TABLES AND FIGURES

APPENDIX E

EMBANKMENT PERMEABILITY

E1 INTRODUCTION

This Appendix summarizes the data available regarding the permeability of the core.

E2 DATA FROM DESIGN REPORT AND TECHNICAL PAPERS

Table 7.4.1 of the Design Report No H1756 lists the "estimated permeabilities based on field and laboratory testing to June 1966 as:

- Core $2x10^{-8}$ m/sec
- Transition $1x10^{-6}$ m/sec
- Pervious Upstream Shell $1x\ 10^{-1}$ m/sec
- Random Upstream Shell $2x10^{-7}$ m/sec

A footnote to the table indicates that "Subsequent (1987) re-evaluation of available test results suggest actual permeability values to be about one order of magnitude higher for each zone."

Table 7.4.1 is the same as Table 1 in Morgan and Harris (1967). That paper indicates that the permeabilities were obtained from triaxial consolidation tests and from tests in an 8 inch dia. (200mm dia.) permeameter.

E3 DATA ON PERMEABILITY OF THE CORE FROM CONSTRUCTION RECORDS

At the request of the EEP BC Hydro has processed the construction test data into construction years and location and elevation in the core. These are the data from laboratory consolidation and permeameter tests, and from in-situ tests in test pits and wells in the embankment core. The results are summarized in Table E4 at the back of this Appendix.

From these it can be noted that:

(a) There is a large difference between permeabilities calculated from consolidation tests and permeameter tests on the same samples. The permeameter results are nearly two orders of magnitude greater.
(b) The median permeability of the permeameter data is about 1.5 x 10^{-6} m/sec and the range is 3.1 x 10^{-8} m/sec to 4 x 10^{-6} m/sec.
(c) The consolidation test data is all from 1965. The median permeability from RL 1675 ft to RL 1825 ft is about $1x10^{-7}$ m/sec and from RL 1825 ft to RL 1900 ft about 2 x 10^{-8} m/sec., or 5 times lower.
(d) The median of the well permeability and Chasi permeability tests in pits and Observation Bores are 1.8 x 10^{-7} m/sec and 2.7 x 10^{-7} m/sec respectively with an overall range of 7.0 x 10^{-8} m/sec to 8.8 x 10^{-7} m/sec.
(e) There is relatively little data.

E4 FALLING HEAD TESTS IN SONIC DRILL HOLES DURING 1996 INVESTIGATIONS

E4.1 Undisturbed core

The results of falling head tests carried out in the core during the sonic drilling are summarized in Table 5-2 of Report MEP 399.

Drill holes DH 96-27, DH 96-31, DH 96-32 and DH 96-33 are all at least 67 ft or 20m from the sinkholes so should be representative of the as-placed core.

There are a total of 20 tests. The summary of these is:

Average permeability 5×10^{-7} m/sec

Lowest 25% of tests 1 to 2×10^{-7} m/sec

Highest 25% of tests 1 to 5×10^{-6} m/sec

E4.2 Core in the highly and moderately disturbed zones around the benchmarks

Drill holes DH 96-34 and DH 96-10 and DH 96-11 are located about 8ft, 7ft and 12 ft respectively from Benchmarks 1 and 2 and are in what has been described in MEP 399 as highly disturbed and moderately disturbed core.

There are 7 tests. The summary of these is:

Average permeability 2×10^{-5}m/sec
Lowest 2 tests 1 to 7×10^{-6} m/sec
Highest 2 tests 3 to 6×10^{-5} m/sec

E4.3 Transition zone

There was only one test giving a permeability of 4×10^{-6}m/sec

E5 CPTU DISSIPATION TESTS IN THE CORE DURING THE 1996 INVESTIGATIONS

The permeability can be estimated from the CPTU dissipation tests. This relies on the calculated CH values which are mostly about 3 cm^2 /minute for undisturbed core and in the range of 10 to 50 cm^2/minute, with some up to 500 cm^2 /minute, in the disturbed area around the sinkholes. This gives permeabilities of about 1.0×10^{-10} m/sec in the undisturbed core, and 10^{-7} m/sec in the disturbed core.

These are so much lower than permeabilities measured by other methods they seem to be unrealistic.

E6 LABORATORY PERMEABILITY FROM 2003 INVESTIGATIONS

Appendix B in Report E107, WAC Bennett Dam Deficiency Investigation, Report on Air Theory and Fines Migration" has data which relates the saturated permeability of the core to the degree of saturation at the time of compaction, and the relationship between the saturated permeability to partially saturated permeability. It includes data from Laval University, Quebec, and Thurber Engineering.

The tests were carried out on a single particle size distribution which is shown in Figure E1.

The soil plots in the coarsest 20% of the gradations from construction records.

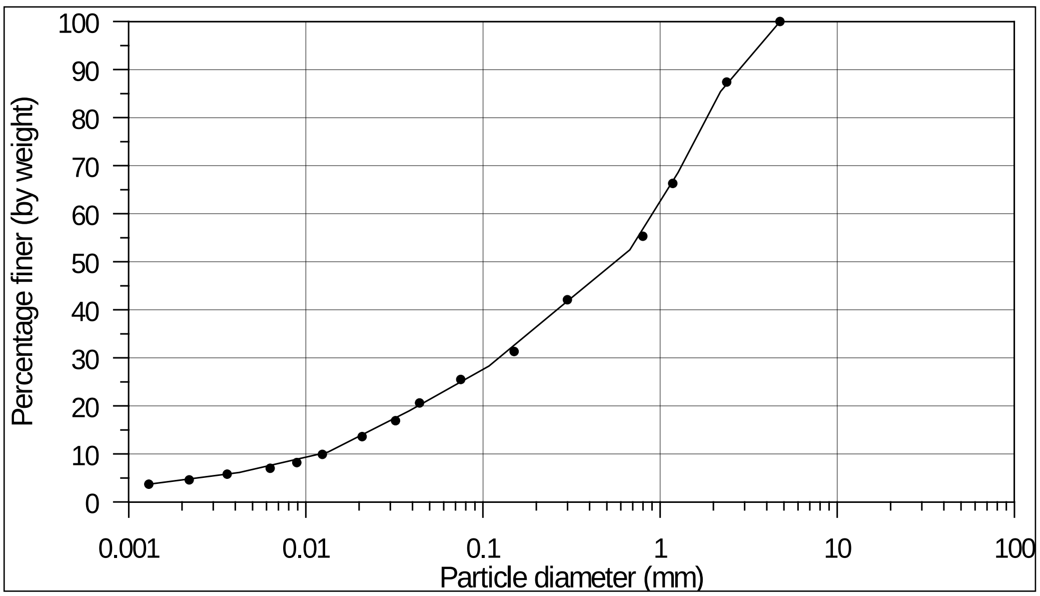

Figure E1. Particle size distribution plot for core sample tested by University Laval and Thurber Engineering in 2003

Laval University carried out permeability tests in an axial and in a radial direction. The axial tests were back pressure saturated before the permeability was measured. The radial tests seem to have been started at the compaction moisture conditions but should represent essentially saturated permeabilities. The Thurber tests were also done without specific attempts to saturate the soil.

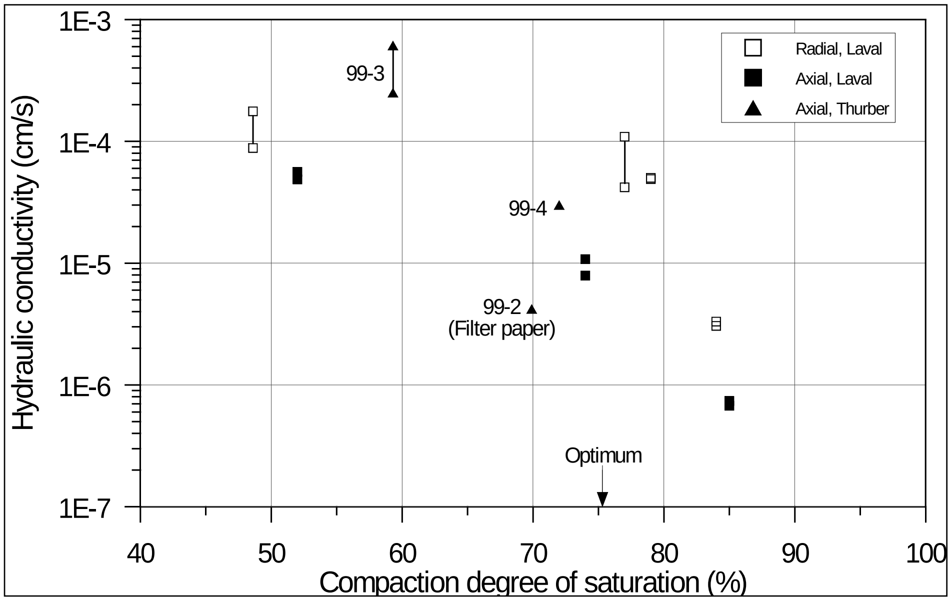

Figure E2. Saturated permeability (hydraulic conductivity) as a function of degree of saturation of the soil at compaction for soil tested by Laval University and Thurber Engineering in 2003.

Figure E2 shows the results of the tests. The report indicates that one Thurber test was affected by clogging of filter paper and should be disregarded. It points out the marked reduction in permeability of samples compacted wet of standard compaction optimum moisture content. They note that this has been observed in tests on similar glacial till soils reported in Watabe et.al. (2000). The significant change in permeability relates to the structure of the soil.

(Note that this is important when considering the University of British Columbia Tests und Jonathan Fannin. They place the soils by consolidating from a slurry so almost certainly create a far lower permeability than the fill in the dam, and may create soils less susceptible to internal erosion)

Suction tests were also carried out to allow definition of the relationship between the unsaturated permeability and the saturated permeability. The tests were done by first saturating a sample compacted wet of optimum moisture content, and then de-saturated by increasing the air pressure.

This gave the following relationship:

k = ksat $S_r^{\alpha\kappa}$

where k is the permeability of the unsaturated soil, ksat is the permeability of the saturated soil, S_r is the degree of saturation, and $\alpha\kappa$ is a relative permeability parameter which for this soil was 6.5.

This was similar to the Watabe et al (2000) soil. This means that the unsaturated permeabilities are significantly lower than the saturated permeability particularly at low degrees of saturation when compacted.

The importance of this information can be seen when the large variations in the degree of saturation during placement of the fill shown in Figure E3 and E4 are considered. These Figures were prepared by BC Hydro from the construction records at the request of the EEP. Figure E5 shows the degree of saturation data for 1965 in more detail.

It can be seen that the degree of saturation varies between about 30% and 100%, and quite commonly from 45% to 85% almost on a daily basis.

Applying this range to Figure E2, University of Laval tests, the permeability for soils placed at varying degrees of saturation are as shown in Table E1.

Table E1. Saturated permeability of core versus degree of saturation at placement based on Laval University axial tests

Degree of saturation when placed in the dam %	Saturated permeability	
	Axial Tests m/sec	Radial Tests m/sec
85	7×10^{-9}	2×10^{-8}
76	9×10^{-8}	8×10^{-7}
60	3×10^{-7}	1×10^{-6}
45	1×10^{-6}	1.2×10^{-6}

It can be seen that there is a significant difference between the axial and radial values probably reflecting some layering in the samples during compaction. The radial values are probably a better representation of horizontal permeability in the field.

The ratio of the saturated permeabilities at 45% and 85% degree of saturation at placement is between about 60 and 140.

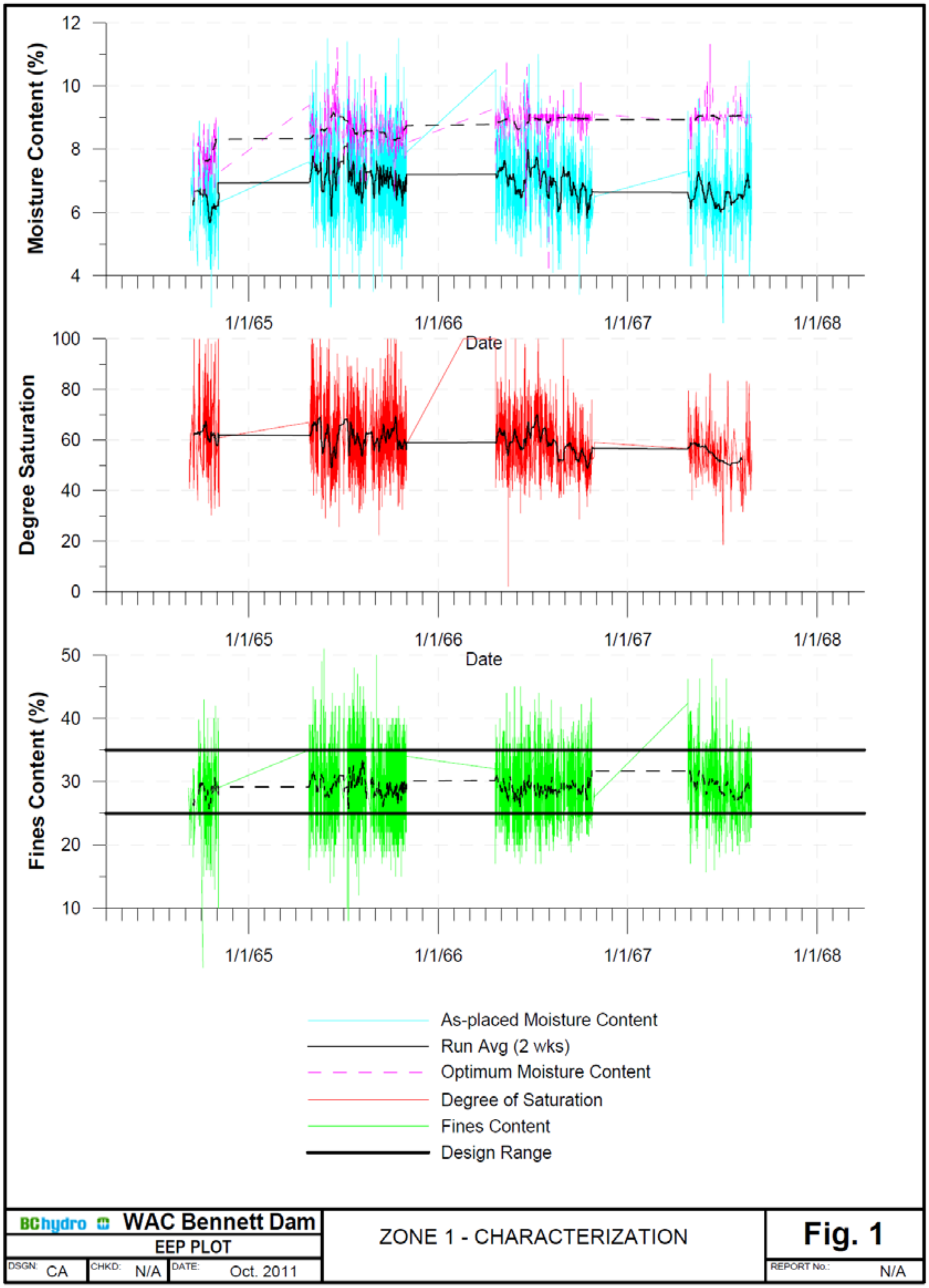

Figure E3. Moisture content, degree of saturation and fines content of Zone 1 core from construction records.

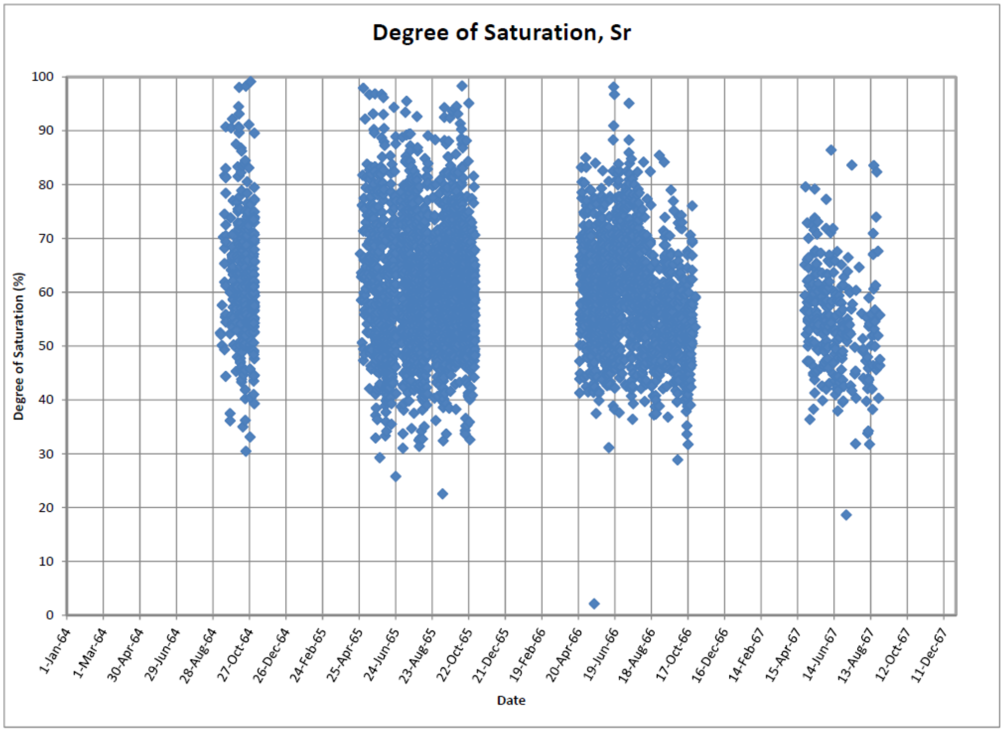

Figure E4. Degree of saturation versus time from construction records.

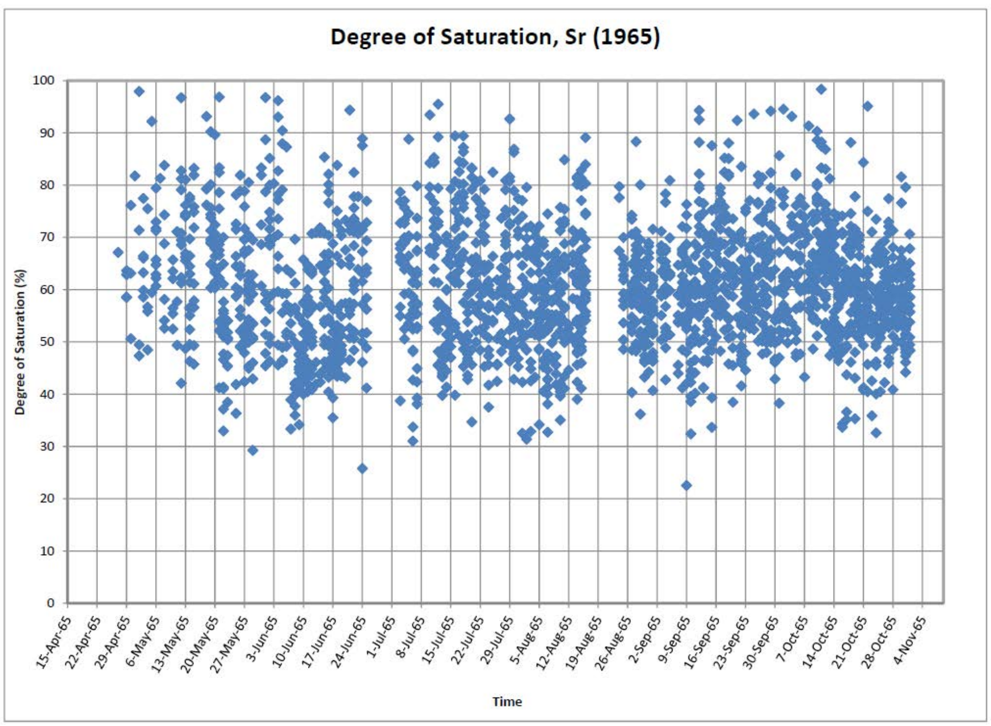

Figure E5. Degree of saturation versus time in 1965 from construction records.

Taking the distribution of degree of saturation from Figure E3, and the saturated permeabilities from Figure E2 the following weighted horizontal permeabilities and Kh/Kv values can be estimated:

1964 Kh = 1 x 10^{-6}m/sec; Kh/Kv = 100

1965 Kh = 1 x 10^{-6}m/sec; Kh/Kv = 100

1966 Kh = 1 x 10^{-6}m/sec; Kh/Kv = 10

1967 Kh = 2 x 10^{-6}m/sec; Kh/Kv = 2

E7 DATA FROM PIEZOMETER OBSERVATIONS DURING FIRST FILLING

E7.1 Approximate Analysis

There are hand plots of the piezometer readings in the dam Core on first filling of the dam. From these it is possible to calculate the rate at which the wetting front moved through the core. Table E2 summarizes this information.

Making approximate allowance for the average gradient between piezometers as the wetting front progresses, and assuming the porosity = 0.22 based on the average compaction dry density of 128 pcf and soil particle density of 2.68, the back-calculated permeability on first filling is around 5x 10^{-6} m/sec.

Table E2. Wetting Front Velocities in the Core on First Filling

Piezometer(s)	Elevation ft	Response time days	Horizontal distance from upstream face of core, ft	Wetting front velocity ft / day
Instrument Plane 1				
EP 05, 06	1795	87	130	(1.5)
EP 07	1795	103	238	6.75
EP 08	1795	131	350	4
EP 12, EP 13	1925	25	85	(3.4)
EP 14	1925	80	230	2.6
EP 16, EP 17	2045	35	50	(1.4)
EP 18	2045	No response		
Instrument Plane 2				
EP 60, EP 61	1865	33	180	(5.4)
EP 62	1865	62	280	3.4
EP 63	1865	90	380	3.6
EP64, EP 65	1955	87	80	(0.9)
EP 66	1955	87	170	Very high
EP 69, EP 70	2030	55	50	(0.9)
EP 71	2030	No response	125	

Note. Figures in brackets are based on zero time for wetting front to travel through upstream shell.

There are quite varying wetting velocities for the upstream piezometers assuming no time for the wetting front to move through the upstream shell. Most are lower than for the wetting front velocity between piezometers indicating that there has been a finite wetting front time in the shell. The piezometers installed in the Upstream Shell took some time to respond to the reservoir level but then showed piezometer pressures very close to reservoir level indicating the Upstream Shell is high permeability compared to the Core.

E7.2 BC Hydro Transient Seepage Analysis

The EEP requested that BC Hydro carry out a transient seepage analysis to refine the approximate estimates in Section E7.1.

This is reported in "Transient Seepage Modelling during Initial Reservoir Filling", Report GMS11DSD Dated September 2011.

Three cases were modelled:

(a) "As designed" homogeneous, anisotropic core, with saturated hydraulic conductivity of $K_h = 1.0 \times 10^{-8}$ m/sec, $K_v = 2.0 \times 10^{-9}$ m/sec.
(b) "Dry of optimum" homogeneous, isotropic core with saturated hydraulic conductivity of $K_h = 1 \times 10^{-6}$ m/sec and $K_v = 2.0 \times 10^{-7}$ m/sec
(c) "As-designed core with construction defects" (winter horizons, lead trenches, instrument islands) with Core saturated hydraulic conductivity of $K_h = 1.0 \times 10^{-8}$ m/sec, $K_v = 2.0 \times 10^{-9}$ m/sec; defects saturated hydraulic conductivity of $K_h = 1 \times 10^{-6}$ m/sec and $K_v = 2.0 \times 10^{-7}$ m/sec. This model also allowed for some clogging of the transition.

A number of assumptions were made to model the unsaturated soil properties. These can be regarded as reasonable but very approximate.

The first of these models grossly underestimated the recorded pore pressures having only the upstream part of the core saturated when the reservoir has reached EL 2132, when in fact all piezometers were recording positive pressures.

The second model also underestimates the rapidity of the rate of saturation of the core and underestimates the pore pressures significantly.

The third model comes closest to modelling the recorded pressures. This seems to be mostly because the defects are modelled with a saturated hydraulic conductivity of 10^{-5} m/sec, ten times that for the first model.

The EEP requested that a fourth case be modelled:

Homogeneous an isotropic core, with saturated hydraulic conductivity of $K_h = 5 \times 10^{-6}$ m/sec and $K_v = 5.0 \times 10^{-6}$ m/sec, and the grout blanket and curtain $K_h = K_v = 3.9 \times 10^{-7}$ m/sec, or 3 lugeons.

This resulted in somewhat better modelling than the first two models with the wetting front matching the actual wetting front advance, but pore pressures are underestimated in the lower parts of the embankment as the reservoir fills.

E8 DISCUSSION ON PERMEABILITY OF THE CORE

E8.1 Undisturbed Core

Table E3 summarizes the available data. The following comments are made:

(a) In-situ tests are generally more reliable than laboratory tests because they test a larger sample and can therefore test the effects of layering and segregation. However all are “flow-in” tests so may under-estimate the permeability if particles block the sides of the test section.
(b) The back-analysis of the first filling is in effect a large scale field test and can be considered as a relatively reliable source of data. There might be an argument that the wetting front has travelled preferentially along the trenches in which the piezometer leads are placed and the backfill in the trenches is more permeable. The argument against this is that the piezometer readings seem to be consistent after reaching their early readings rather than dropping if the surrounding core was at a lower pore pressure.
(c) The consolidation test related data are probably the least reliable as they rely on back-analysis of consolidation data. They seem to be an order or more, lower permeability than the other tests.
(d) It would be expected that there would be layering of permeability within the core due to the effects of variable particle size distribution from different parts of the borrow areas and times from the production plant, differential compaction at the top and bottom of layers, variable moisture content and degree of saturation on compaction. This is in fact clearly demonstrated in the data in Figure E3 and in the laboratory test data carried out in 2003.
(e) The effects of the degree of saturation during compaction on the saturated permeability is large as shown in Table E1. The horizontal permeability will be controlled by the low degree of saturation layers, and the vertical by the high degree of saturation layers. These occur throughout the dam as shown in Figure E3.
(f) The laboratory and in-situ testing does not in individual tests model the layering within the core due to variations in particle size distribution, degree of saturation on compaction, segregation and differential compaction within layers. The CPT testing near Benchmarks 1 and 2 beyond the immediate low density zones show distinct layering at about 5 ft intervals in some CPT, e.g. CPT 98-3 and CPT 96-4.

Based on this information a reasonable estimate of the average properties of the undisturbed core would be:

1964	$K_h = 2 \times 10^{-6}$ m/sec; $K_h/K_v = 100$	
1965	$K_h = 2 \times 10^{-6}$ m/sec; $K_h/K_v = 100$	
1966	$K_h = 2 \times 10^{-6}$ m/sec; $K_h/K_v = 10$ (First half of season, to end of July)	
	$K_h = 4 \times 10^{-6}$ m/sec; $K_h/K_v = 2$ (Second half of season from 1st August)	
1967	$K_h = 4 \times 10^{-6}$ m/sec; $K_h/K_v = 2$	

This allows for the marked differences in the degree of saturation in the years of construction. For the second part of 1966 and 1967 it appears that there was more control over compaction moisture contents with all fill being placed at degrees of saturation below that for optimum moisture content. Hence from Figure E2 the permeabilities are similar throughout.

Table E3. Summary of permeability data for undisturbed core

Source of Information	Type of Testing	Permeability data		
		Median	Maximum	Minimum
Design Report and Technical Papers	"Field and Laboratory Testing"	$2x10^{-8}$ m/sec		
Construction Records Permeameter	Laboratory	$1.5x10^{-6}$ m/sec	$4x10^{-6}$m/sec	$3x10^{-8}$ m/sec
Construction Records Consolidation	Laboratory	$1x10^{-7}$ m/sec RL 1675 ft to RL 1825 ft 2 $x10^{-8}$ m/sec. RL 1825 ft to RL 1900 ft	$1x10^{-6}$ m/sec $4.5x10^{-8}$ m/sec	$1.4x10^{-8}$ m/sec. $1.4x10^{-8}$ m/sec
Construction Records Well permeability	In-situ	$1.8x10^{-7}$ m/sec	$5.3x10^{-7}$ m/sec	$4.4x10^{-8}$ m/sec
Construction Records Chasi	In-situ	$2.7x10^{-7}$ m/sec	$8.8x10^{-7}$ m/sec	$7x10^{-8}$ m/sec
Falling Head in Sonic Core Holes	In-situ	$5x10^{-7}$ m/sec	5 x 10^{-6} m/sec	1 x 10^{-7} m/sec
Laboratory permeability from 2003 investigations.	Laboratory Radial (Kh) Axial (Kv)	1 x 10^{-6} m/sec 3 x 10^{-7} m/sec	1.2 x 10^{-6} m/sec 1 x 10^{-6} m/sec	2 x 10^{-8} m/sec 7 x 10^{-9} m/sec
Back-analysis of First Filling Simplified analysis	Based on "In-situ" Data	5x 10^{-6} m/sec.		
Transient Analysis of First Filling		Kh > 10^{-6} m/sec.		

E8.2 Core surrounding the benchmarks

The data in Section E4.2 is probably a good guide to this. These zones are almost certainly much more permeably than the core generally.

E9 REFERENCES

Morgan G.C. and Harris M.C. (1967) Portage Mountain Dam II Materials. Canadian Geotechnical Journal, Vol IV, No 2, 142-183.

Watabe, Y., Leroueil. S, and Le Bihan, J-P. (2000) Influence of compaction conditions on pore size distribution and saturated hydraulic conductivity of a glacial till. Canadian Geotechnical Journal Vol 37, 1184-11

Table E4. Permeability Data from Construction records

Sample #	DATE / YEAR	LOCATION			PLACEMENT CONDITIONS			PERMEABILITY		
		STATION	OFFSET	ELEV.	Compaction %	Moisture Content %	Opt Moisture Content %	K (cm/sec)	Test Type	Remarks
E1-350	17-Oct-64	4900	175 U	1693		7.2		1.04E-05	Consolidation	
E1-550	28-Oct-64	5050	375 U	1692		5.5		1.08E-05	Consolidation	
E1-1100	6-May-65	Left Abut	25 U	1708		7.2		3.28E-06	Consolidation	
E1-1300	20-May-65	5300	175 U	1720		7.4		9.86E-06	Consolidation	
E1-1500	1-Jun-65	5575	25 D	1745		6.7		6.7	Consolidation	
E1-1700	11-Jun-65	4950	375 U	1730		6.7		6.7	Consolidation	
E1-2300	13-Jul-65	4430	390U	1759		5.6		9.61E-05	Consolidation	
E1-2700	E1-2700	4900	15U	1807		5.6		1.46E-05	Consolidation	
E1-2900	3-Aug-65	4550	250U	1800				8.60E-06	Consolidation	
E1-3100	9-Aug-65	4410	200U	1803		4.1		1.43E-06	Consolidation	
E1-3300	14-Aug-65	5550	15U	1830		4.4		3.51E-06	Consolidation	
E1-3500	29-Aug-65	4080	400U	1824		4.7		1.22E-05	Consolidation	
E1-3700	8-Sep-65	4250	200U	1835		7.5		1.15E-06	Consolidation	
E1-3900	13-Sep-65	5625	20D	1855		7.8		3.65E-06	Consolidation	
E1-4300	26-Sep-65	5525	70U	1877		6.2		1.37E-06	Consolidation	
E1-4500	2-Oct-65	5450	0	1889		7.0		4.56E-06	Consolidation	
E1-4700	8-Oct-65	5000	225U	1865		4.8		3.60E-06	Consolidation	
E1-4900	12-Oct-65	4500	50D	1867		4.9		2.60E-06	Consolidation	
E1-5100	26-Oct-65	5350	25U	1900	100.6	6.5		1.53E-04	8" Permeameter	
						6.0		2.33E-06	Consolidation	
E1-5300	20-Oct-65	5200	100U	1885	99.1	7.9		1.17E-04	8" Permeameter	
						5.6		1.67E-06	Consolidation	
E1-5600	26-Oct-65	3650	350U	1870	95.4	7.7		2.14E-04	8" Permeameter	
E1-5700	28-Oct-65	3430	10D	1876		5.2		1.42E-06	Consolidation	
E1-5900	1-Nov-65	3525	25D	1885	98.5	6.5		2.19E-04	8" Permeameter	
						7.5		1.67E-06	Consolidation	

Sample #	DATE / YEAR	LOCATION			PLACEMENT CONDITIONS			PERMEABILITY		
		STATION	OFFSET	ELEV.	Compaction %	Moisture Content %	Opt Moisture Content %	K (cm/sec)	Test Type	Remarks
E1-6200	10-Jun-66	5050	20 D	1927	98.8	6.6		1.72E-04	8" Permeameter	
E1-6500	20-Jun-66	5100	0	1937	102.1			2.15E-05	8" Permeameter	
E1-6800	28-Jun-66	5200	180 U	1955	103.7	5.3		1.25E-04	8" Permeameter	
E1-7100	16-Jul-66	4850	40 D	1963		6.6		6.92E-05	8" Permeameter	
E1-7400	30-Jul-66	4200	175 U	1975	99.2	7.5		3.25E-04	8" Permeameter	
E1-7700	9-Aug-66	5100	125 U	1986	99.1	7.7		8.32E-05	8" Permeameter	
E1-8000	25-Aug-66	3300	30 D	2010	99.0	7.2		2.20E-04	8" Permeameter	
E1-8300	14-Sep-66				100.2	6.3		3.08E-05	8" Permeameter	
E1-8900	15-Oct-66	2600	10 D	2095	98.9	5.7		3.14E-06	8" Permeameter	
E1-10246	19-May-67	5800	20 D	2052	96.2	7.0		3.97E-05	8" Permeameter	
E1-10400	6-Jun-67	5650	30 D	2070	95.0	7.6		1.49E-04	8" Permeameter	
E1-10617	28-Jun-67	5825	50 U	2088	96.0	6.3		4.01E-04	8" Permeameter	
E1-10807	24-Jul-67	6600	25 D	2120	100.2	5.8		2.79E-04	8" Permeameter	
E1-10998	16-Aug-67	5600	10 D	2175	97.0	9.2		7.20E-06	8" Permeameter	
TP1-H	Nov-65							2.70E-05	Falling head	Test Pit Horizontal
TP2-V	Dec-65							1.90E-05	Falling head	Test Pit Vertical
TP3-H	Jan-66							2.70E-05	Constant head	
TP3-V	Feb-66							6.00E-05	Constant head	
TP4-H	Mar-66							1.50E-05	Constant head	
TP4-V	Apr-66							7.00E-06	Constant head	
O.W.-H	May-66							8.80E-05	Constant head	Observation well
O.W.-V	Jun-66							1.00E-05	Constant head	
O.W.-H	Jul-66							3.40E-05	Falling head	

Sample #	DATE / YEAR	LOCATION			PLACEMENT CONDITIONS			PERMEABILITY		
		STATION	OFFSET	ELEV.	Compaction %	Moisture Content %	Opt Moisture Content %	K (cm/sec)	Test Type	Remarks
Well Perm. Test 66-1	Jun-66	5757	20 D	1954				5.30E-05		
Well Perm. Test 66-2	Jun-66	2775	75 U	1956				2.80E-05		
Well Perm. Test 66-4	Jul-66	5876	28 U	1980				4.40E-06		
Well Perm. Test 66-5	Jul-66	2780	58 U	1969				5.40E-06		
Well Perm. Test 66-7	Jul-66	3366	175 D	1971				4.70E-05		
Well Perm. Test 66-9	Aug-66	5753	99.15 U	1988				1.80E-05		

Appendix F: The Bennett Dam Filter System and its Effectiveness in Arresting Internal Erosion, including the Effects of Internal Instability

APPENDIX F

THE BENNETT DAM FILTER SYSTEM AND ITS EFFECTIVENESS IN ARRESTING INTERNAL EROSION, INCLUDING THE EFFECTS OF INTERNAL INSTABILITY

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FIGURES

APPENDIX F

THE BENNETT DAM FILTER SYSTEM AND ITS EFFECTIVENESS IN ARRESTING INTERNAL EROSION, INCLUDING THE EFFECTS OF INTERNAL INSTABILITY

F1 INTRODUCTION

This Appendix discusses the filter system at Bennett Dam and its effectiveness in arresting internal erosion.

It includes a discussion of the effects of internal instability and suffusion of internally unstable soils, as this affects the filter system.

Figure F1 (at the back of this Appendix) shows a plan of the dam with the location of the two Instrument Planes, and Survey Benchmarks 1 and 2 at which sinkholes developed in 1996. Figure F2 shows the location of site investigations.

F2 DESCRIPTION OF THE FILTER SYSTEM

Figures F3 and F4 show the zoning at Instrument Planes 1 and 2 (IP1, IP2).

The filter system as constructed for Bennett Dam consists of:

Transition (Zone 2) which is a broadly graded gravelly sand and sandy gravel, with between 2% and 8% fines passing 0.075mm.

Filter (Zone 3) which is sandy gravel with less than 5% fines passing 0.075mm.

Drain (Zone 4) which is gravel with some to a trace of sand, and less than 2% fines passing 0.075mm.

The Transition may be considered as a fine filter and the Filter as a coarse filter.

The critical elevations for internal erosion are:

Dam Crest	2230 ft
Top of Core	2220 ft
Top of Filter	2190 ft
Top of Drain/change in core section	2160 ft
Normal Maximum reservoir level	2205 ft
Normal Minimum reservoir level	2100 ft
PMF Flood Level current operating	2207 ft, Spring flood; 2208 ft, Summer flood.

Above EL 2190, the Transition is the single line of filter defence. The Transition also acts as the drain.

The PMF levels are taken from an extract of the Report on Dam Safety, Williston Lake, Probable Maximum Flood, dated May 2010. This report indicated that the current operating rules aim to keep the reservoir level as low as practicable because of the concerns after the 1996 sinkhole event.

The report indicates that under the old operating rules in 1988 the PMF reached a level of 2215 ft. This difference is important as the old PMF levels would take the reservoir closer to the dam crest level and into areas more likely subject to cracking due to differential settlements during construction.

Figures F5 to F8 show the as-constructed gradations for the Core, Transition, Filter and Drain. These are plots of every 5th test carried out during construction. It will be seen that the gradations vary from year to year. Figure F9 shows a selected typical gradation for the Upstream Random Fill.

The Transition and Upstream Random Fill are typically somewhat gap graded with a deficiency of coarse sand and fine gravel. The Filter has a somewhat different shaped gradation and is not gap graded.

The ability of the Transition and Filter to arrest erosion in the Core is affected by the gradation as placed, but also by the potential for suffusion to occur under leakage flows with selective removal of part or the entire finer fraction resulting in a coarser gradation.

Figure F10 shows what is meant by “finer” and “coarser” fractions.

The gradations in Figures F5 to F9 were taken from density in place tests. For Zones 2, 3, 4 and 6 these were in 30 inch (750mm) diameter rings and sampled up to 15 inches (225mm) depth (Low and Lyell, 1967). This means that the affects of segregation of the materials as they were placed may not be reflected in the gradations. The Zone 3 Filter samples were also taken at three locations across the zone and combined so are to an extent averaged gradations.

These factors need to be taken account of when considering their effectiveness as filters.

F3 METHOD OF ASSESSMENT OF THE EFFECTIVENESS OF THE FILTER SYSTEM

F3.1 Allowing for the effects of Suffusion

The effects of suffusion have been assessed by using the method of Wan (2006), Wan and Fell (2004, 2008) which adapts the Burenkova (1993) method to allow the probability of a soil being internally unstable and hence subject to suffusion. Wan (2006) carried out tests on silt-sand-gravel soils similar to the transition and filter and is applicable to those materials.

BC Hydro has used the Kenny and Lau (1985, 1986) method to assess internal instability. This method was developed for less broadly graded soils than the transition and filter at Bennett Dam but the results are included for comparison. In their method Kenny and Lau (1985, 1986) indicate that for widely graded materials the limiting amount of finer fraction for internally unstable soils is 20%. That is the controlling factor for much of the Bennett Dam materials.

Wan and Fell found that for the 20 soils they tested Kenny and Lau (1985, 1986) predicted 19 of the 20 soils would be internally unstable when in fact only 9 were under the downwards gradient of 8 used in the tests. They concluded that the Kenny and Lau (1985, 1986) method is conservative for broadly graded silt-sand-gravel soils.

If a soil is subject to suffusion some or all the finer fraction may be eroded from the soil under the leakage flows in the dam. Wan (2006), Wan and Fell (2006) found that for the soils they tested about half of the finer fraction eroded but the actual amount was dependent on the soil gradation. For this report two amounts have been used: (a) Assuming 50% removed; and (b) assuming 100% finer fraction removed. The latter is almost certainly conservative. The method for determining the grading after loss of 50% of the finer fraction is shown in Figure F11.

F3.2 Allowing for the effects of segregation

Broadly graded soils such as the Transition and Filter are subject to segregation as they are placed, with the coarser particles separating from the finer particles and collecting at the base of each layer.

Tests carried out by Kenney et al (1993) and Sherard (1985) on the susceptibility of soils to segregation have shown sand-gravel filters with sand contents (<4.75mm) less than about 40% are particularly susceptible to segregation during handling. Most of the transition at Bennett has > 40% sand < 4.75mm size but about half of the filter has less than 40% sand.

Low and Lyell (1967) indicate that “The filter material is a blended granular material which has proved difficult to manufacture in the plant and to place adequately without excessive segregation.” They also indicate that segregation was a problem for the drain material. From this it can be taken that those involved in construction were aware of the issue and did take steps to overcome them.

Other factors to consider include:

(a) The Transition is about 10 metres wide at the top of Filter level and greater than 20 metres wide at the base of the canyon. At the top of Core it is about 6m wide. The transition was placed parallel to the dam axis (Low and Lyell) and so each layer would involve at least two truck loads, and mostly four or more. This lessens the likelihood of a continuous segregated layer across the transition. The filter is narrower, and may have been placed in one truck width.

(b) The sonic drill core logs and the core for DH96-38 do not show significant segregation. However the drilling method resulted in typically greater lengths of core than drilled length so there may have been some mixing.

From these factors it is considered that segregation was possible and at worst could result in separation of the plus 4.75mm sizes from the finer fraction. To see if this is critical the filter capability of the Transition assuming this complete separation has been checked even though it is an unlikely scenario.

Figure F12 shows an example of how the gradation of the transition or filter has been adjusted to allow for segregation. This is a conservative approach as it assumes that the coarser fraction separates completely from the finer fraction.

F3.3 Filter Criteria

Most dam engineers use gradation based criteria to design filters. These were first developed by Terzhaghi and rely on the D85 of the base soil, and the D15 of the filter. Sherard and Dunnigan (1989) refined these criteria. Their criteria are widely used and are summarized in Table F1. These are known as no-erosion criteria although in fact they may rely on a small amount of erosion of the base soil to create the self filtering mechanism on the soil-filter contact.

Foster (1999), Foster and Fell (2001) used the Sherard and Dunnigan (1989) test data and their own tests to refine the no-erosion criteria and to develop excessive and continuing erosion boundaries. These are summarized in Table F2. Filters which fall between the no-erosion and excessive erosion criteria will experience some erosion before erosion of the base soil is arrested. Those falling between the excessive and continuing erosion criteria will experience excessive erosion. Filters which are coarser than the continuing erosion criteria will not arrest erosion of the base soil.

Foster and Fell (2001) found that information from case histories of poor filter performance suggests the potential maximum leakage flows that could develop due to piping are as follows:

– Filters falling into the Some Erosion category – up to 100 l/sec before sealing

– Filters falling into the Excessive Erosion category – 100 to 1000 l/sec before sealing

– Filters falling into the Continuing Erosion category – flows of 1000 l/sec and increasing.

These criteria were developed with case data including many dams with glacial core materials and are considered appropriate for use with Bennett materials.

Table F1: No erosion boundary for the assessment of filters of existing dams (Foster and Fell 2001)

Base Soil Category	Fines content (1)	Design Criteria of Sherard and Dunnigan (1989)	Range of DF15 for No Erosion Boundary From Tests	Criteria for No Erosion Boundary
1	≥ 85%	DF15 ≤ 9 DB85	6.4 - 13.5 DB85	DF15 ≤ 9 DB85 (2)
2	40 - 85%	DF15 ≤ 0.7 mm	0.7 - 1.7 mm	DF15 ≤ 0.7 mm (2)
3	15 - 40%	DF15 ≤ (40-pp% 0.075 mm) x (4DB85-0.7)/25 + 0.7	1.6 - 2.5 DF15 of Sherard and Dunnigan design criteria	DF15 ≤ (40-pp% 0.075 mm) x (4DB85-0.7)/25 + 0.7
4	< 15%	DF15 ≤ 4 DB85	6.8 - 10 DB85	DF15 ≤ 4 DB85

Notes: (1) The fines content is the % finer than 0.075 mm after the base soil is adjusted to a maximum particle size of 4.75 mm.

(2) For highly dispersive soils (Pinhole classification D1 or D2 or Emerson Class 1 or 2), it is recommended to use a lower DF15 for the no erosion boundary. For soil group 1 soils, suggest use the lower limit of the experimental boundary, i.e. DF15 ≤ 6.4 DB85. For soil group 2 soils, suggest use DF15 ≤ 0.5 mm. The equation for soil group 4 would be modified accordingly.

Table F2: Excessive and Continuing erosion criteria (Foster 1999; Foster and Fell 1999, 2001)

Base Soil	Proposed Criteria for Excessive Erosion Boundary	Proposed Criteria for Continuing Erosion Boundary
Soils with DB95<0.3 mm	DF15 > 9 DB95	For all soils: DF15 > 9DB95
Soils with 0.3<DB95<2 mm	DF15 > 9 DB90	
Soils with DB95>2 mm and fines content >35%	DF15 > the DF15 value which gives an erosion loss of 0.25g/cm^2 in the CEF test (0.25g/cm^2 contour line in Figure A below)	
Soils with DB95>2 mm and fines content <15%	DF15 > 9 DB85	
Soils with DB95>2 mm and fines content 15-35%	DF15 > 2.5 DF15 design, where DF15 design is given by: DF15 design=(35-pp%0.075 mm)(4DB85-0.7)/20+0.7	

Notes: Criteria are directly applicable to soils with DB95 up to 4.75 mm. For soils with coarser particles determine DB85 and DB95 using grading curves adjusted to give a maximum size of 4.75 mm.

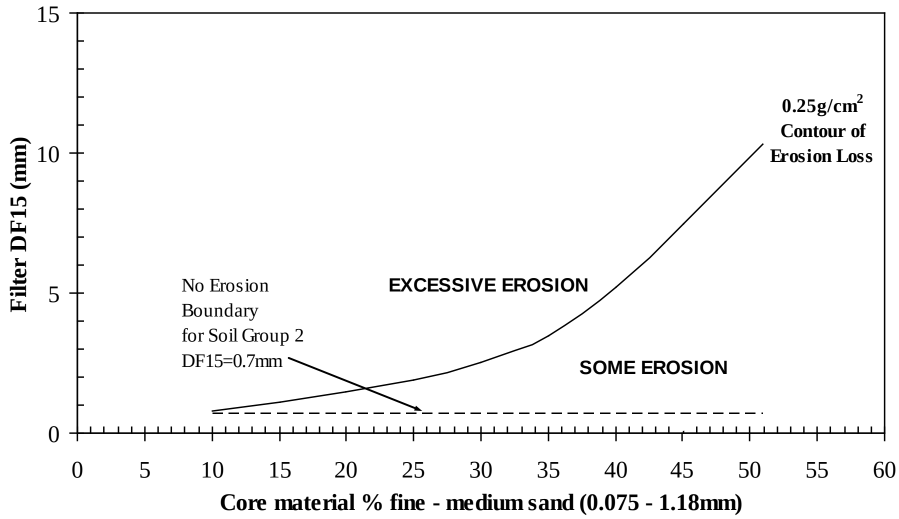

Figure A Criteria for Excessive Erosion Boundary.

F3.4 Likelihood of filters holding a crack

Transitions and filters which have a percentage of non-plastic fines passing 0.075mm may hold a crack and not perform as required. This is dependent on the percentage of fines, the degree of compaction and the effects of mineralogy, particularly cementing. Table F3 is from Fell et al (2008) and is based on research carried out by Park (2003) and case data. It is used to assess the likelihood of holding a crack for concentrated leak erosion where the mechanism causing a crack in the core may also form a crack in the transition.

Table F3: Likelihood for Filters with Excessive Fines Holding a Crack (Fell et al, 2008)

Fines Plasticity	Fines Content % Passing 0.075 mm	Probability of holding a crack	
		Compacted	Not compacted
Non plastic (and no cementing present)	5%	0.001	0.0002
	7%	0.005	0.001
	12%	0.05	0.01
	15%	0.1	0.02
	>30%	0.5	0.1
Plastic (or fines susceptible to cementing)	5%	0.05	0.02
	7%	0.1	0.05
	12%	0.5	0.3
	≥ 15%	0.9	0.7

Note: Fines susceptible to cementing for filters having a matrix predominately of sand sized particles (e.g. filters derived from crushed limestone).

F4 ASSESSMENT OF FILTER SYSTEM AS CONSTRUCTED

F4.1 Assessment of the likelihoods of internal instability

F4.1.1 Internal instability of the Core

Table F4 summarizes the likelihood of internal instability of the Core. It can be seen that it is very unlikely that any part of the Core is internally unstable.

Table F4: Assessment of the likelihood of internal instability of the Core

Construction Year	Wan and Fell method Probability of internal instability		Kenny and Lau Method Percentage of gradations susceptible (BC Hydro estimates)
	Overall Assessment	Worst gradation	
1964	>98% negligible likelihood	0.02	2.2
1965	>98% negligible likelihood	0.02	0.7
1966	100% negligible	Negligible	0.4
1967	100% negligible	Negligible	1.4

F4.1.2 Internal instability of the Transition

Table F5 summarizes the likelihood of internal instability of the Transition.

Table F5: Assessment of the likelihood of internal instability of the Transition

Construction Year	Wan and Fell method Probability of internal instability		Kenny and Lau Method Percentage of gradations susceptible (F < 20%)
	Overall Assessment	Worst gradation	
1964	20% > 0.3 50% > 0.02	0.4	23.5
1965	20% > 0.3 50% > 0.05	0.7	15.1
1966	10% > 0.5 50% > 0.1	0.7	6.7
1967	10% > 0.5 50% > 0.1	0.7	7.0

Note. Wan and Fell method probabilities are approximate as they have been done manually.
Overall it is assessed that it is likely that about 20% of the Transition has a high likelihood of being internally unstable and therefore subject to suffusion, and about half of the Transition some chance of internal instability. These may be over-estimates as the Wan (2006) testing was carried out under low stresses and the materials may be more resistant to suffusion under high stresses.

F4.1.3 Internal instability of the Filter

Table F6 summarizes the likelihood of internal instability of the Filter.

Table F6: Assessment of the likelihood of internal instability of the Filter

Construction Year	Wan and Fell method Probability of internal instability		Kenny and Lau Method Percentage of gradations susceptible (F < 20%)
	Overall Assessment	Worst gradation	
1964	20% > 0.1	0.25	63.4
1965	70% > 0.5	0.6	78.6
1966	70% > 0.5	0.6	73.3
1967	70% > 0.5	0.6	78.5

Note. Wan and Fell method probabilities are approximate as they have been done manually.
From this it assessed that much of the Filter is internally unstable. It was less so for the 1964 year of construction.

F4.1.4 Internal instability of the Drain

Table F7 summarizes the likelihood of internal instability of the Drain.

Table F7: Assessment of the likelihood of internal instability of the Drain

Construction Year	Wan and Fell method Probability of internal instability		Kenny and Lau Method Percentage of gradations susceptible (F < 30%)
	Overall Assessment	Worst gradation	
1964	90% > 0.1	0.15	59.1
1965	90% > 0.2	0.25	98.2
1966	90% > 0.2	0.25	97.4
1967	90% > 0.2	0.25	94.6

Note. Wan and Fell method probabilities are approximate as they have been done manually.

The Wan and Fell method was not calibrated on drain type gradations and the Kenny and Lau method is regarded as more applicable.

From this it assessed that most of the Drain is internally unstable. It was less so for the 1964 construction.

F4.1.5 University of British Columbia tests on Core and Transition to assess internal instability.

Permeameter tests done at UBC on samples determined to be representative of Core and Transition from Bennett Dam. These are reported in Moffat et al (2011) and Moffat and Fannin (2011). Figure F13 shows the gradations of the samples tested.

The samples were placed as a slurry and consolidated in the permeameter. They were subject to vertical down and vertical up flows with de-aired water.

The consolidated samples were between 325mm and 550mm long. The samples were supported on a wire mesh of 2.76mm opening for the Transition and 0.15mm for the Core. Using the relation that opening size = D_{15} /9 these are equivalent to D_{15} sizes of 25mm for the

Transition samples and 1.35mm for the Core samples. These are far coarser than the as-placed D_{15} which were between 0.2mm and 0.7mm for the Transition and between 0.25mm and 2.5mm for the Filter.

The Core samples which had a gradation on the coarse side of the as-placed Core gradations exhibited episodic movement of small quantities of finer fraction until the gradient was 27 and 29, at which time there was sudden "energetic boiling action" throughout the samples.

The Transition samples had a gradation approximately at the centre of the as-placed gradations. The gradation of these samples was more horizontal in the "gap" than any of the actual gradations. Using the Wan and Fell (2004, 2008) method the probability of internal instability for that gradation was 0.02.

Each increment of increase in gradient caused washout of relatively fine particles at the mesh supporting the sample. The erosion stopped with time. No movement of particles was observed within. The samples did not exhibit significant erosion until the gradient was 11 in one test and 31 in the second. At this stage there was a loss of volume.

The Authors referred to the episodic movement of fines as suffusion, and the boiling action as suffosion. From the description the "suffusion" was minor and stopped after a short time. Given the test set up the suffusion may have been backward erosion rather than suffusion from within the sample. The "boiling" action seems to be indicative of hydraulic fracturing of the sample followed by concentrated leak erosion.

These tests add weight to the evidence that the Core is internally stable, and that the Transition, with low likelihood of instability by the Wan and Fell method, is internally stable.

However the tests were not representative of the most potentially internally unstable Transition. The use of a consolidation procedure to place the soil may give a soil structure and permeability less likely to test as internally unstable.

F4.2 Assessment of the capability of the Transition as a filter to the Core

Table F8 summarizes the assessed capability of the Transition as a filter to the Core. The table leads to the following conclusions:

(a) From the as-placed gradations from during construction, **all** the Transition is a no-erosion filter to **all** the as constructed gradations from the Core.

(b) If 50% loss of finer fraction is allowed for in those Transition materials which are internally unstable and subject to suffusion, there is potential for some erosion of the Core but the Transition will arrest erosion with only minor leakage.

(c) If 100% erosion of the finer fraction is allowed for in those Transition materials which are unstable and subject to suffusion, there is potential for some erosion of the Core in all years of construction, and a lesser likelihood of excessive erosion in 1965 and 1966 materials. If excessive erosion occurred somewhat larger leakage may occur before the Transition arrests erosion.

(d) If complete segregation of the coarser fraction from the finer fraction of the Transition occurs, which is a very unlikely scenario, there is potential for some to excessive erosion in 1964 and 1965 materials, and excessive erosion in 1966 and 1967 materials.

(e) There is no scenario of segregation or suffusion of the Transition which can lead to continuing erosion, so the **Transition should arrest erosion of the Core for all plausible scenarios**.

F4.3 Assessment of the capability of the Filter as a filter to the Transition

Table F9 summarizes the assessed capability of the Filter as a filter to the Transition. From this:

(a) From the as-placed gradations from during construction, all the Filter is a no-erosion filter to all the as constructed gradations from the Transition.

(b) If 50% loss of finer fraction is allowed for in those Filter materials which are internally unstable and subject to suffusion, the Filter is still a no-erosion filter to the Transition.

(c) If 100% erosion of the finer fraction is allowed for in those Filter materials which are unstable and subject to suffusion, there is potential for some erosion of the Transition in all years of construction. There are no situations where this would result in excessive erosion.

(d) If complete segregation of the coarse fraction from the finer fraction of the Filter occurs, which is a very unlikely scenario, there is potential for some erosion of the Transition in all years of construction. There are no situations where this would result in excessive erosion.

(e) There is no scenario of segregation or suffusion of the Filter which can lead to excessive or continuing erosion, so the Filter should arrest erosion of the Transition with minor leakages for all plausible scenarios.

(f) There is potential for some movement of the finer fraction of the Transition within itself as suffusion, and into the Filter as a “some erosion” scenario.

Table F8: Assessed Filter Capability - Transition to Core

Year	Core Gradations			Required Transition Gradations			Actual Transition Gradations D15		
	% Fines after regrading	DB85 after regrading	DB95 after Regrading	No Erosion DF15	Excessive Erosion DF15	Continuing Erosion DF15	As placed	Coarsest after suffusion (1)	Coarsest segregation (2)
1964	43 to 28	1.6 to 2.5	3.2 to 3.8	0.7 to 5	4 to 10	28 to 34	0.2 to 0.5	1.4 to 2.5	8
1965	45 to 22	1.2 to 2.2	2.8 to 3.5	0.7 to 10	5 to 15	25 to 32	0.2 to 0.5	2.2 to 8	8
1966	42 to 28	0.85 to 1.8	1.2 to 2.5	0.7 to 3.8	2.3 to 8	11 to 22	0.2 to 0.35	1.2 to 5	12
1967	52 to 25	0.6 to 1.8	2.0 to 3.3	0.7 to 4.6	6 to 10	18 to 30	0.2 to 0.7	1.8 to 3	12

Notes: (1) The figures are for 50% and 100% of the finer fraction eroded. The latter is an unlikely scenario.
(2) This assumes complete segregation of the coarse fraction from the finer fraction. It is a very unlikely scenario.

Table F9: Assessed Filter Capability – Filter to Transition

Year	Transition Gradations			Required Filter Gradations			Actual Filter Gradations		
	% Fines after regrading	DB85 after regrading	DB95 after Regrading	No Erosion DF15	Excessive Erosion DF15	Continuing Erosion DF15	As placed	Coarsest after suffusion (1)	With segregation (2)
1964	6 to 3	2.5 to 3	3.8 to 4.1	10 to 12	22 to 27	34 to 37	0.25 to 2.5	0.6 to 6	13
1965	10 to 7	1.6 to 2.2	3.0 to 3.8	6.4 to 8.8	14 to 20	27 to 34	0.4 to 1.9	1.6 to 12	14
1966	10 to 5	1.6 to 1.7	1.9 to 2.0	6.4 to 6.8	15 to 16	17 to 18	0.4 to 1.5	1.5 to 12	13
1967	12 to 9	1.3 to 2.8	2.5 to 4.0	4.2 to 11.2	12 to 25	22 to 36	0.4 to 1.5	2.0 to 12	13

Notes: (1) The figures are for 50% and 100% of the finer fraction eroded. The latter is an unlikely scenario.
(2) This assumes complete segregation of the coarse fraction from the finer fraction. It is a very unlikely scenario.

Table F10: Assessed Filter Capability – Drain to Filter

Year	Filter Gradations			Required Drain Gradations			Actual Drain Gradations		
	% Fines after regrading	DB85 after regrading	DB95 after Regrading	No Erosion DF15	Excessive Erosion DF15	Continuing Erosion DF15	As placed	After suffusion	With segregation
1964	14 to 5	2.7 to 3.3	4.0 to 4.3	11 to 13	24 to 30	36 to 39	6 to 13	7 to 13	15
1965	6 to 3	2.5 to 2.9	4.0 to 4.2	10 to 12	22 to 26	36 to 38	2.5 to 16	7 to 17	17
1966	7 to 3	2.4 to 3.3	3.7 to 4.2	10 to 13	22 to 30	33 to 38	5 to 22	8 to 24	24
1967	5 to 3	2.0 to 2.9	3.6 to 4.1	8 to 12	18 to 26	32 to 37	5 to 22	8 to 24	24

Notes: (1) The figures are for 50% and 100% of the finer fraction eroded. The latter is an unlikely scenario.
(2) This assumes complete segregation of the coarse fraction from the finer fraction. It is a very unlikely scenario.

Table F11: Assessed Filter Capability – Drain to Zone 6 (Transition) at Splitter Dikes No 2 and No 4.

Year	Transition Gradations			Required Drain Gradations			Actual Drain Gradations		
	% Fines after regrading	DB85 after regrading	DB95 after Regrading	No Erosion DF15	Excessive Erosion DF15	Continuing Erosion DF15	As placed	After suffusion	With segregation
Splitter Dike No 2, 1965									
1965	10 to 7	1.6 to 2.2	3.0 to 3.8	6.4 to 8.8	14 to 20	27 to 34	2.5 to 16	7 to 17	17
Splitter Dike No 4, 1967									
1967	12 to 9	1.3 to 2.8	2.5 to 4.0	4.2 to 11.2	12 to 25	22 to 36	5 to 22	8 to 24	24

Notes: (1) The figures are for 50% and 100% of the finer fraction eroded. The latter is an unlikely scenario.
(2) This assumes complete segregation of the coarse fraction from the finer fraction. It is a very unlikely scenario.

F4.4 Assessment of the capability of the Drain as a filter to the Filter

Table F10 summarizes the assessed capability of the Drain as a filter to the Filter. From this:

(a) From the as-placed gradations from during construction about 90% of the Drain is a no-erosion filter to the Filter in 1964, and about 75% in 1965, 1966 and 1967. The remainder of the Drain satisfies excessive erosion criteria as placed except for one gradation in 1967. For these there may be some erosion before the Drain arrests erosion from the Filter.

(b) If 50% loss of finer fraction is allowed for in those Drain materials which are internally unstable and subject to suffusion, the Drain is still a no-erosion or some- erosion filter to the Transition.

(c) If 100% erosion of the finer fraction is allowed for in those Drain materials which are unstable and subject to suffusion, there is potential for some erosion of the Filter in all years of construction. There are a small number of situations in 1966 and 1967 where this would result in excessive erosion.

(d) If complete segregation of the coarse fraction from the finer fraction of the Drain occurs, which is a very unlikely scenario, there is potential for some erosion of the Filter in all years of construction and a small number of situations in 1966 and 1967 where this would result in excessive erosion.

(e) There is no scenario of segregation or suffusion of the Drain which can lead to continuing erosion, so the Drain should arrest erosion of the Filter with minor leakages for all plausible scenarios.

(f) There is potential for some movement of the finer fraction of the Filter within itself as suffusion, and into the Drain as a “some erosion” scenario.

F4.5 Potential for Erosion of the Core into the Splitter Dike and Random Fill adjacent the Splitter Dike

Figures 7-1, 7-2 and 7-3 in Report MEP 400 show the details of Splitter Dikes No 2 and No. 4 which are located close to Benchmarks 1 and 2 and have been identified in some reports and papers as potentially being a factor in the development of the Sinkholes at the Benchmarks.

The Dikes are constructed of Zone 6, Random Fill. Adjacent to Splitter Dike No 4 there is some Zone 6 material placed adjacent the Splitter Dike which abuts the core (Figs 7-2 and 7-3).

In Report MEP 402, Section 4.8.8 it is pointed out that there is no filter between Splitter Dike No 2 and the Drain. It is stated that the Drain does not provide a filter for the shell material (Zone 6), and that if the fine portion of the Zone 6 is internally eroded a path for piping of the core through the Zone 6 to the Drain could develop.

In Sections 6.2.1, 6.2.2, 6.2.3 and 6.2.4 this is discussed further.

From Figure 6 of Low and Lyell (1967) it seems that in 1965 the Random Shell material had virtually the same 80% gradation boundaries as the Transition. Coarser material was allowed in the Random Shell but it seems not used that year.

At the Panel’s request the database of particle gradations was searched for gradations of Zone 6 in the vicinity of Splitter Dike No 4. There were only two gradations as shown in Figure F14. These are within the boundaries of Transition material placed in 1967. One is in about the coarsest 1/3rd of the Transition, and the second on the fine boundary.

From these data it is concluded that Zone 6 had gradations within the range of those for the Transition, so the critical filtering for Zone 6 to Core will be as for Transition to Core.

Table F 11 summarizes filtering between the Drain and Transition for the years that Splitter Dikes No 2 and No 4 were constructed.

From this it can be seen that:

(1) The Drain is generally too coarse to be a no-erosion filter to the Transition and therefore to the Zone 6, in 1967 when Splitter Dike No 4 was constructed. In 1965 when Splitter Dike No 2 was constructed there was a greater likelihood the Drain was a no-erosion filter.

(2) There is a low likelihood that the Drain would be coarser than an excessive erosion filter to the Transition / Zone 6. Hence erosion of the Transition / Zone 6 should be arrested with some erosion. Some gradations at the limits give slightly coarser than excessive erosion but significantly finer than the continuing erosion criteria.

From these it can be concluded that the Splitter Dikes are no more likely to lead to erosion of the core than the Transition in general, and that the fact that Zone 6 is placed adjacent to the Drain does not increase the likelihood of internal erosion leading to significant leakage flows.

F5 ASSESSMENT OF FILTER SYSTEM AS SHOWN BY SITE INVESTIGATIONS IN 1996-1997

F5.1 Transition to Core

Figures F15 to F17 present gradations of the Core and Transition on samples taken from the boreholes drilled in 1996 and 1997.

From these it can be seen that:

(1) The gradations for the Core are within the range of gradations for the as-placed Core. In particular there are no finer gradations.

(2) The gradations for the Transition are within the range of gradations for as placed Transition except that there are some finer gradations such as those shown in Figure F17. These gradations may be a result of breakdown of coarse particles due to the sonic drilling. This is discussed further in Section F7.

(3) The coarsest gradation in the Transition in DH 96-38 shown in Figure F17 is on the coarse boundary of the as-placed gradations.

In view of (1) and (3), there is nothing from the borehole samples which is worse from a filter compatibility viewpoint than discussed above for the as-placed Core and Transition.

F5.2 Filter to Transition

There is no data from boreholes for the Filter so the gradations for the Transition from the boreholes must be compared to the as-placed Filter gradations.

When the finest borehole Transition sample (as shown on Figure F17) is re-graded on the 4.75mm sieve it has a D85 of 1.6mm. As can be seen from Table F9 this is within the range of D85 for the as-placed Transition for 1965, so even though it is a lot finer than the as-placed transition the Filter will still be effective in arresting erosion if the gradation is real and not affected by the sonic drilling process.

F6 ASSESSMENT OF THE LIKELIHOOD THE TRANSITION WILL HOLD A CRACK

The upper part of the embankment above El 2160 ft has no Zone 3 Filter so the filter system relies only on the Zone 2 Transition to arrest erosion in the core.

For concentrated leak erosion it is necessary to consider whether the mechanism causing a crack in the core may cause a crack in the adjacent Transition, and if water flows through the crack, whether the Core and Transition will hold the crack. If the Transition does hold a crack it may not perform its filtering function.

This can be assessed using Table F3 and the percentage fines from the as-placed gradations for the Transition. Table F12 summarizes the outcomes.

Table F12. Likelihood of Core and Transition holding a crack based on as constructed gradations

Construction Year	Core		Transition	
	% Fines Passing 0.075mm	Likelihood of holding a crack	% Fines Passing 0.075mm	Likelihood of holding a crack
1964	20 to 38	0.2 to 0.6	1 to 6	Negligible to 0.001
1965	19 to 45	0.2 to 0.9	1 to 7	Negligible to 0.005
1966	23 to 42	0.2 to 0.9	1 to 7	Negligible to 0.005
1967	20 to 52	0.2 to 0.9	3 to 9	Negligible to 0.02

It can be seen that the core is highly likely to hold a crack.

The Transition is unlikely to do so but there is some chance it may do in the upper parts of the dam where higher fines content Transition was used. This may be further investigated by laboratory tests. It is important because the Transition is the only element in the filter system in the upper parts of the dam.

F7 ASSESSMENT OF THE FACTORS WHICH MAY HAVE RESULTED IN THE HIGHER FINES CONTENT FOUND IN THE TRANSITION IN THE 1996 INVESTIGATIONS

Figure F18 shows the location of sonic drill holes into the Transition in the 1996 investigations. Figure F19 shows a plot of fines content versus elevation for these holes.

It can be seen that:

(a) The fines contents from the 1996 investigations are almost all greater than the specified limit. The construction records indicate that the specified limits were achieved during construction.
(b) There are high fines contents above EL 2165. These may however be samples from the Core as the holes were close to the boundary of the Core and Transition.

(c) There are several examples of high fines contents below EL 1900 approx. These are in DH 96-38, which is at Instrument Plane 1, DH 96-17 which is near Instrument Plane 2, and DH 96-28 which is at Benchmark 2.
(d) There are no very high fines contents below the top few feet in DH 96-25 which is at Benchmark 2.

There are a number of possible reasons for these high fines contents:

(1) They are a result of breakdown from to the drilling action of sonic drilling.

(i) Figure F20 shows gradations from samples in borehole DH 96-38, 1800ft to 1900ft. Figure F21 shows these gradations adjusted by regrading so the maximum fines content is 5%, the specified limit. From this it can be seen that there is a deficiency in the coarse sand and gravel sizes, which might occur if the drilling action broke these coarse particles into fines.

(ii) In support of this hypothesis is the fact that this did occur in the drilling of the Drain as shown in Figure F22.

(iii) Figure F23 shows a plot of % fines versus depth for DH96-28, which was drilled using sonic drilling and P96-10 which was drilled using the Barber dill, with samples taken from below the base of the drill hole using a 5 inch inside diameter (125 mm) diameter 10 ft long sampler which was driven into the soil. It can be seen that the fines contents are similar using the two methods. However the Barber drilling technique does not go sufficiently deep to allow a check of the zone in DH96-28 where high fines contents are indicated.

(iv) Also it is noted that the upper elevation of the higher fines contents coincided with changing from 7.6 inch drill casing to 5.5 inch drill casing. The 5.5 inch casing was used with 4 inch or 100mm diameter core barrel. Given that 10% to 20% of the particles are 1 inch or greater size the small barrel almost certainly would not have been able to penetrate the Transition without breaking the particles or pushing them aside. However the 5.5 inch casing was used below about EL 2050 in DH 96-25 and there were no very high fines contents but the average % fines is higher over these elevations in DH 96-25 (11%) compared to DH96-38 (about 8%).

(v) Experience with sonic drilling elsewhere is that even cobble size particles and in-situ rock can be broken to dust by the drilling action.

(vi) At the request of the EEP BC Hydro carried out an analysis of the shape of the fines particles in the high fines content core samples and compared these to the shape of fines particles in samples taken from the South Moraine and silt borrow areas. If particle breakdown was the cause of the higher fines content there should be more angular particles. This work is reported in BC Hydro Memo, WAC Bennett Dam-Zone 2 Transition Fines, dated 30 January 2012. This was inconclusive because a large % of the particles in the moraine are angular, and the analyses were only done on the quartz particles. Mineralogy analysis on a limited number of samples supported the view that the additional fines were derived from the core. The report indicates a larger data set would be required to test this. That report includes plots of particle size distributions adjusted to 5% fines as in Figure F21. However the plots also adjust the coarse sizes by "scalping" off the plus 1.5 inch sizes, so they do not show the reduction in the % coarse sizes as Figure F21 does.

(2) The high fines content may be explained by movement of the finer fraction of the Transition as a suffusion process.

(i) The high fines contents are in the upper 90 ft of the 1965 construction period. From Table F5 Transition placed in 1964 and 1965 is the most susceptible to internal instability by the Kenny and Lau method. The Wan and Fell method also indicates internal instability is likely in the coarser material. However 1964 and 1965 are not too different to the other years by this method. In any case it is quite probable that some of the Transition is internally unstable and subject to suffusion under relatively small hydraulic gradients. If this occurred there would likely be a movement of finer particles from the upstream part of the Transition to the downstream. Typically this is what happens in laboratory tests, the fines are not necessarily eroded from the soil, in this case into the Filter.

(ii) The erosion of finer fraction from the upstream area of the Transition would make the Transition become too coarse to be a no erosion filter. This is shown in Table F8. From Table F8 it can be seen that the 1965 construction gives the coarsest Transition after suffusion of all construction years. If the finer fraction eroded from the Transition the coarsest D15 would be 8mm, and the finest excessive erosion D15 for the core would be 5mm, so falling between the excessive erosion and continuing erosion criteria. The same situation is possible in 1966 construction. This means that some erosion of the core can be expected before the Transition acts as a filter to the core over most of the dam, and in 1965 and 1966 construction, “excessive erosion” might occur in some situations.

(iii) Laboratory backward erosion tests on a marginally internally unstable glacial till soil from a dam in Australia were tested for 47 days before the erosion process progressed to give loss of fines throughout the sample and a sudden increase in flow. The sample was 250mm diameter and 300mm high. The gradation of the sample (the coarse gradation) and the cumulative erosion versus time are shown in Figure F24 and F25. The mesh size on which the sample is seated is important as it replicates the filter. The mesh size of 4.75 mm was selected as it is equivalent to a D_{15} size of about 43 mm. From this it seems that the suffusion process may be quite slow, particularly in a wide transition as at Bennett Dam.

(iv) The multiport piezometers show that some read up to 20 ft positive pressure when installed in 1997. In particular piezometers P7, P9 and P11 which are located in the zone of high fines content at IP1. The fact that positive pressures exist in the Transition indicates it has some layering allowing perched water tables. It tends to support the possibility of high fines contents within the Transition in these areas. The fact that these pressures are small compared to the pressures in piezometers in the Core at the same level indicate that either the pressure is lost in flow through the Core downstream of the Core piezometers, or on the Core / Transition filter interface. A few multiport piezometers react to fluctuations in reservoir pressure. Most of these are near the foundation and may be reacting to the reservoir level fluctuations via the foundation. Others are near winter shutdown surfaces, but others in IP2 are not. No particular trend seems apparent. One multiport piezometer (P3 in DH97-3, Instrument Plane 1, at about RL 1770 ft) is rising with time indicating a non-equilibrium condition. Most multiport piezometers have zero pressure.

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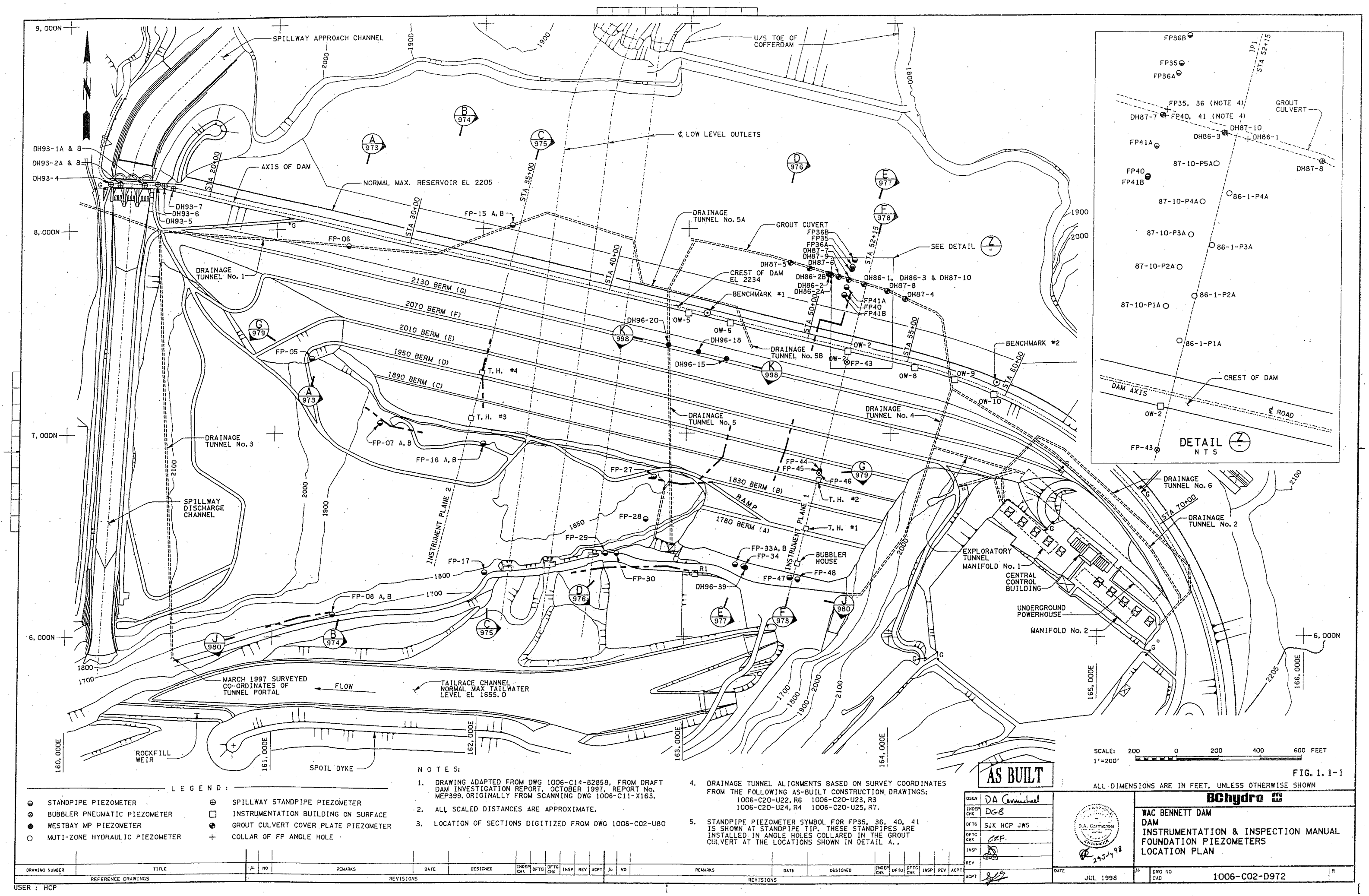
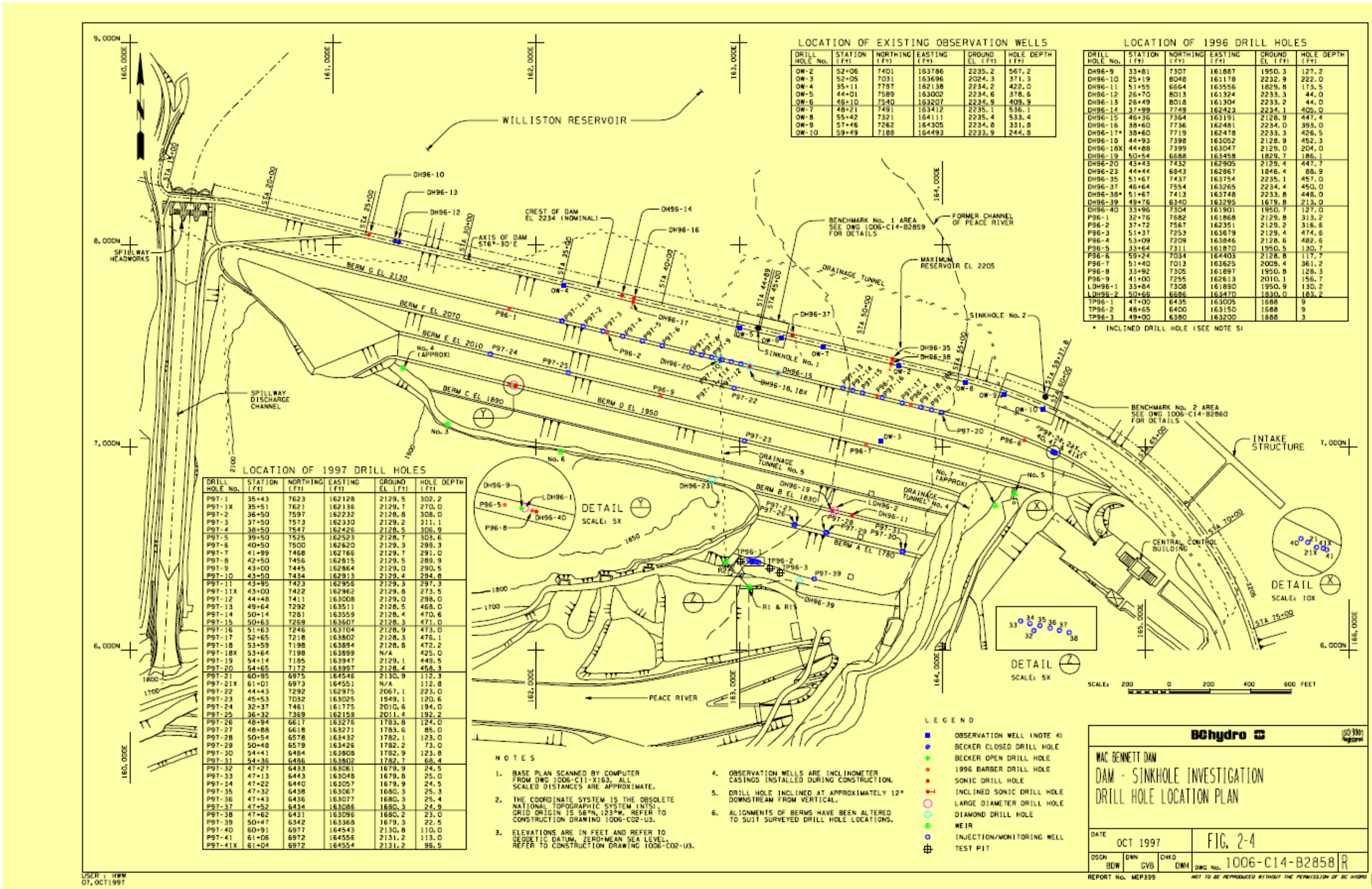


Figure F1: Plan of Bennett Dam showing location of instrument planes, Survey Benchmarks, and Observation Wells



LOCATION OF EXISTING OBSERVATION WELLS

DRILL HOLE No.	STATION (ft)	NORTHING (ft)	EASTING (ft)	GROUND EL (ft)	HOLE DEPTH (ft)
OW-2	52+06	7401	163786	2235.2	567.2
OW-3	52+05	7031	163696	2024.3	371.3
OW-4	35+11	7797	162138	2234.2	422.0
OW-5	44+01	7589	163002	2234.6	378.6
OW-6	46+10	7540	163207	2234.9	409.9
OW-7	48+21	7491	163412	2235.1	536.1
OW-8	55+42	7321	164111	2235.4	533.4
OW-9	57+46	7262	164305	2234.8	331.8
OW-10	59+49	7188	164493	2233.9	244.8

LOCATION OF 1996 DRILL HOLES

DRILL HOLE No.	STATION (ft)	NORTHING (ft)	EASTING (ft)	GROUND EL (ft)	HOLE DEPTH (ft)
DH96-9	33+81	7307	161887	1950.3	127.2
DH96-10	25+19	8048	161178	2232.9	222.0
DH96-11	51+55	6664	163556	1829.8	173.5
DH96-12	26+70	8013	161324	2233.3	44.0
DH96-13	26+49	8018	161304	2233.2	44.0
DH96-14	37+99	7749	162423	2234.1	405.0
DH96-15	46+36	7364	163191	2128.9	447.4
DH96-16	38+60	7736	162481	2234.0	393.0
DH96-17*	38+60	7719	162478	2233.3	426.5
DH96-18	44+93	7398	163052	2128.9	452.3
DH96-18X	44+88	7399	163047	2129.0	204.0
DH96-19	50+54	6688	163458	1829.7	186.1
DH96-20	43+43	7432	162905	2129.4	447.7
DH96-23	44+44	6843	162867	1846.4	88.9
DH96-35	51+67	7437	163754	2235.1	457.0
DH96-37	46+64	7554	163265	2234.4	450.0
DH96-38*	51+67	7413	163748	2233.8	448.0
DH96-39	49+76	6340	163295	1679.8	213.0
DH96-40	33+96	7304	161901	1950.7	127.0
P96-1	32+76	7682	161868	2129.8	313.2
P96-2	37+72	7567	162351	2129.2	316.6
P96-3	51+37	7253	163679	2129.4	474.6
P96-4	53+09	7209	163846	2128.6	482.6
P96-5	33+64	7311	161870	1950.5	130.7
P96-6	59+24	7034	164403	2128.8	117.7
P96-7	51+40	7013	163625	2009.4	361.2
P96-8	33+92	7305	161897	1950.8	128.3
P96-9	41+00	7255	162613	2010.1	156.7
LDH96-1	33+84	7308	161890	1950.9	130.2
LDH96-2	50+66	6686	163470	1830.0	183.2
TP96-1	47+00	6435	163005	1688	9
TP96-2	48+65	6400	163150	1688	9
TP96-3	49+00	6380	163200	1688	3

* INCLINED DRILL HOLE (SEE NOTE 5)

LOCATION OF 1997 DRILL HOLES

DRILL HOLE No.	STATION (ft)	NORTHING (ft)	EASTING (ft)	GROUND EL (ft)	HOLE DEPTH (ft)
P97-1	35+43	7623	162128	2129.5	302.2
P97-1X	35+51	7621	162136	2129.7	270.0
P97-2	36+50	7597	162232	2128.8	308.0
P97-3	37+50	7573	162330	2129.2	311.1
P97-4	38+50	7547	162426	2128.5	306.9
P97-5	39+50	7525	162523	2128.7	303.6
P97-6	40+50	7500	162620	2129.3	299.3
P97-7	41+99	7468	162766	2129.7	291.0
P97-8	42+50	7456	162815	2129.5	289.9
P97-9	43+00	7445	162864	2129.0	290.5
P97-10	43+50	7434	162913	2129.4	294.8
P97-11	43+95	7423	162956	2129.3	297.3
P97-11X	43+00	7422	162962	2129.8	273.5
P97-12	44+48	7411	163008	2129.0	298.0
P97-13	49+64	7292	163511	2128.5	468.0
P97-14	50+14	7281	163559	2128.4	470.6
P97-15	50+63	7269	163607	2128.3	471.0
P97-16	51+63	7246	163704	2128.9	473.0
P97-17	52+65	7218	163802	2128.3	476.1
P97-18	53+59	7198	163894	2128.8	472.2
P97-18X	53+64	7198	163899	N/A	425.0
P97-19	54+14	7185	163947	2129.1	449.5
P97-20	54+65	7172	163997	2128.4	458.3
P97-21	60+95	6975	164546	2130.9	112.3
P97-21X	61+01	6973	164551	N/A	112.8
P97-22	44+43	7292	162975	2067.1	223.0
P97-23	45+53	7032	163025	1949.1	120.6
P97-24	32+37	7461	161775	2010.6	194.0
P97-25	36+32	7369	162159	2011.4	192.2
P97-26	48+94	6617	163276	1783.8	124.0
P97-27	48+88	6618	163271	1783.6	85.0
P97-28	50+54	6578	163432	1782.1	123.0
P97-29	50+48	6579	163426	1782.2	73.0
P97-30	54+41	6484	163808	1782.9	123.8
P97-31	54+36	6486	163802	1782.7	68.4
P97-32	47+27	6433	163061	1679.9	24.5
P97-33	47+13	6443	163048	1679.8	25.0
P97-34	47+22	6440	163057	1679.9	24.5
P97-35	47+32	6438	163067	1680.3	25.3
P97-36	47+43	6436	163077	1680.3	25.4
P97-37	47+52	6434	163086	1680.3	24.9
P97-38	47+62	6431	163096	1680.2	23.0
P97-39	50+47	6342	163368	1679.3	22.5
P97-40	60+91	6977	164543	2130.8	110.0
P97-41	61+06	6972	164556	2131.2	113.0
P97-41X	61+04	6972	164554	2131.2	96.5

Figure F2: Plan of Bennett Dam showing location of drill holes in 1996 investigations

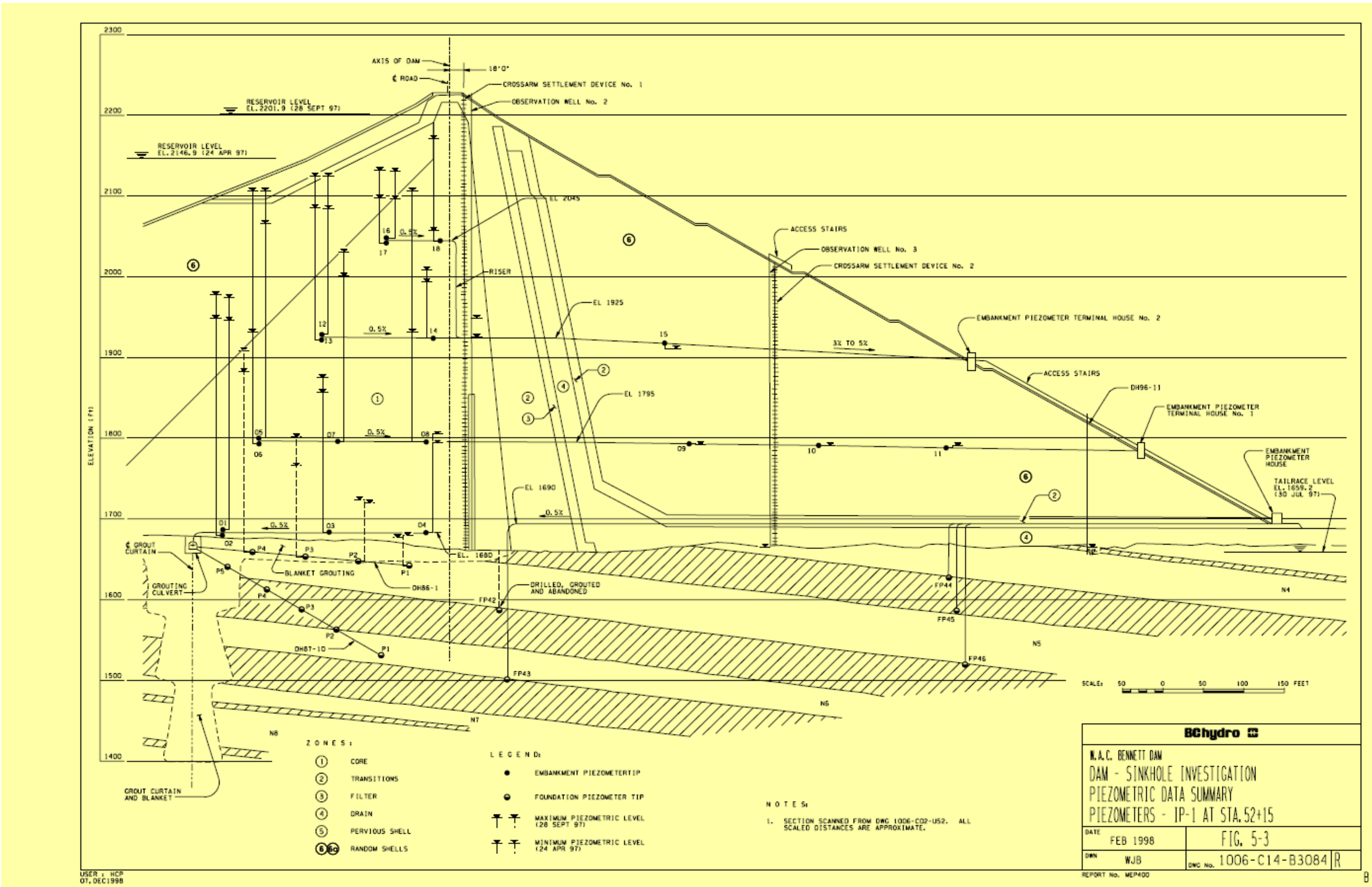


Figure F3: Cross Section - Instrument Plane 1

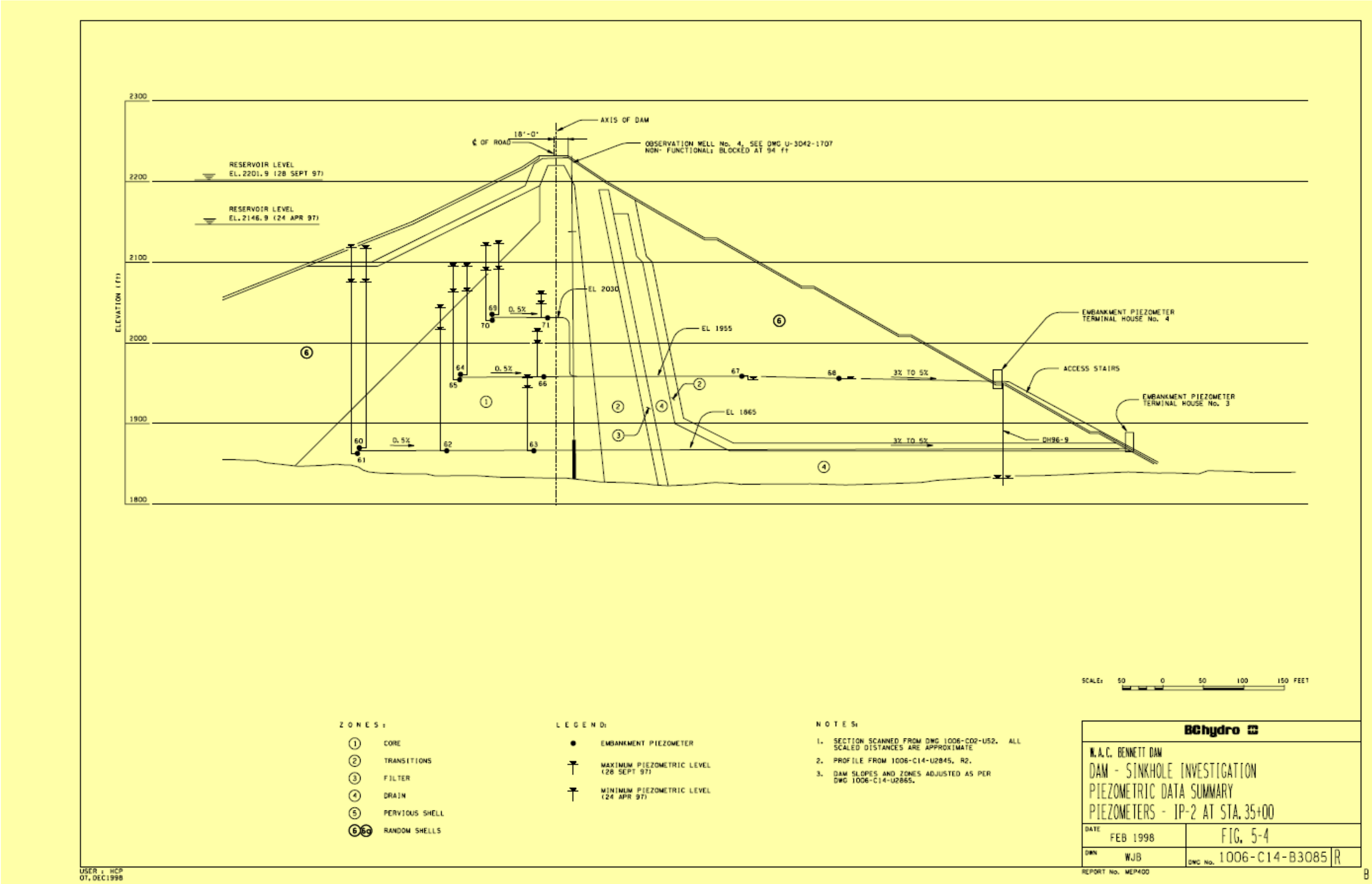
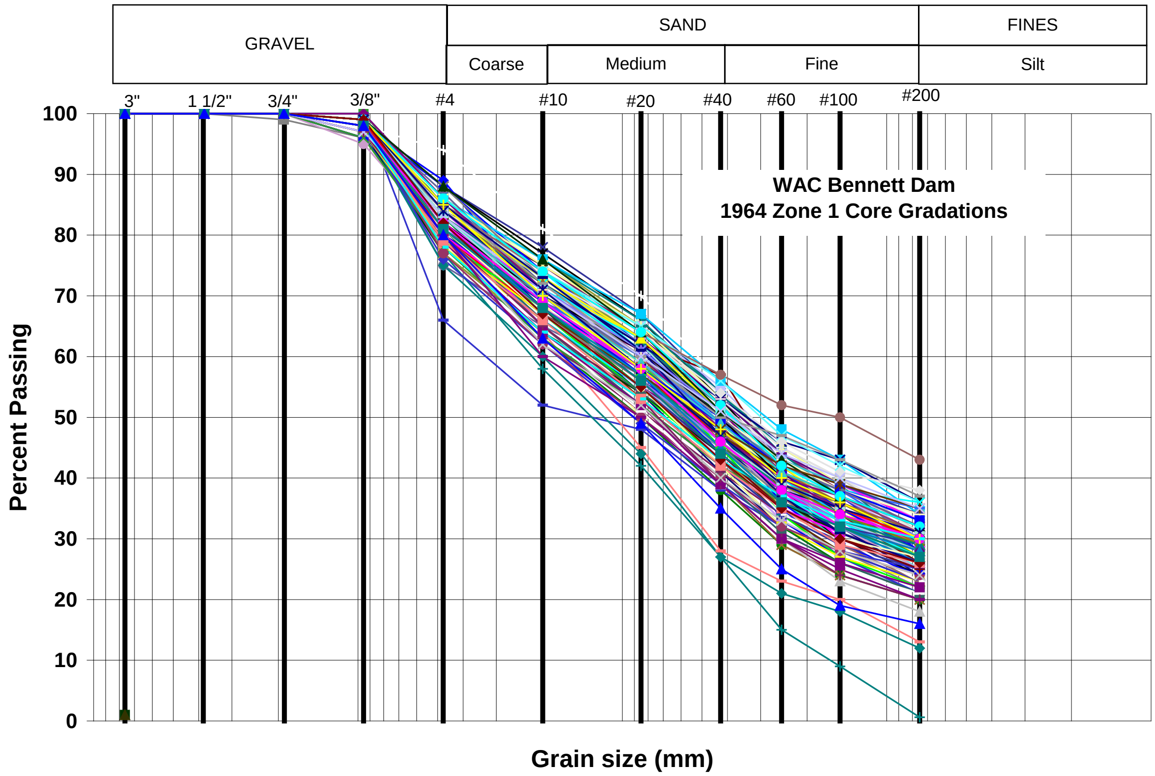


Figure F4: Cross Section- Instrument Plane 2



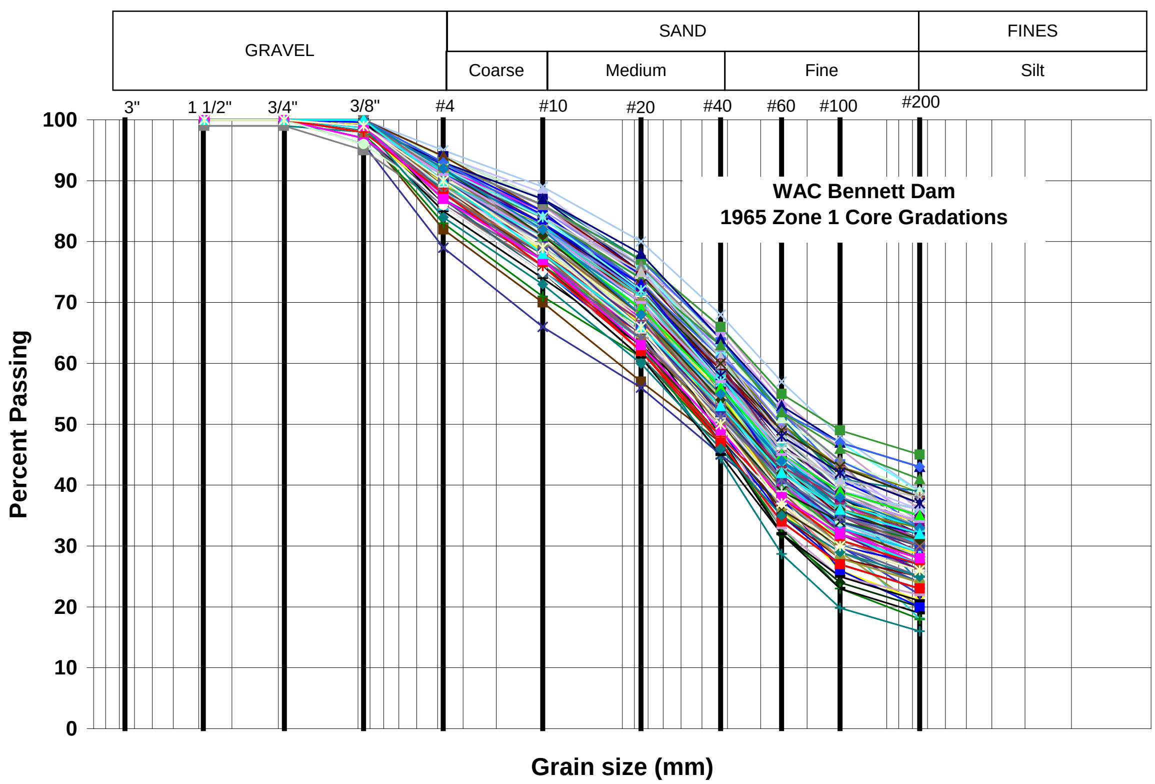
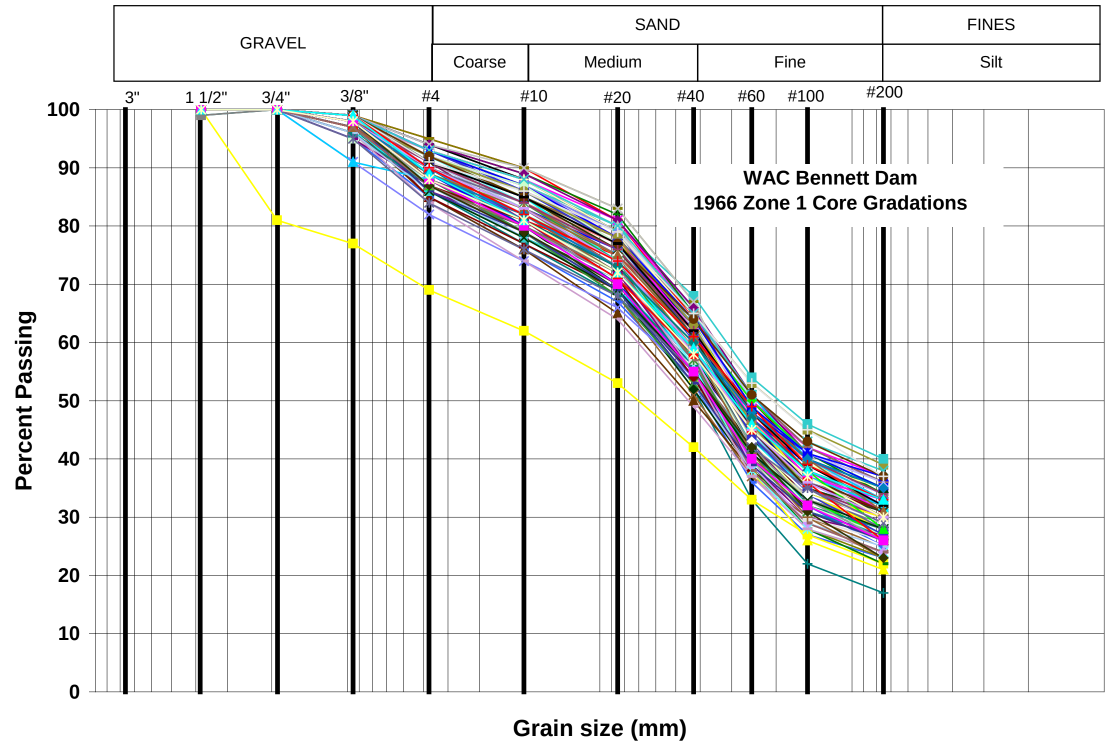


Figure F5a: Gradations of Zone 1 Core from construction records, 1964, 1965



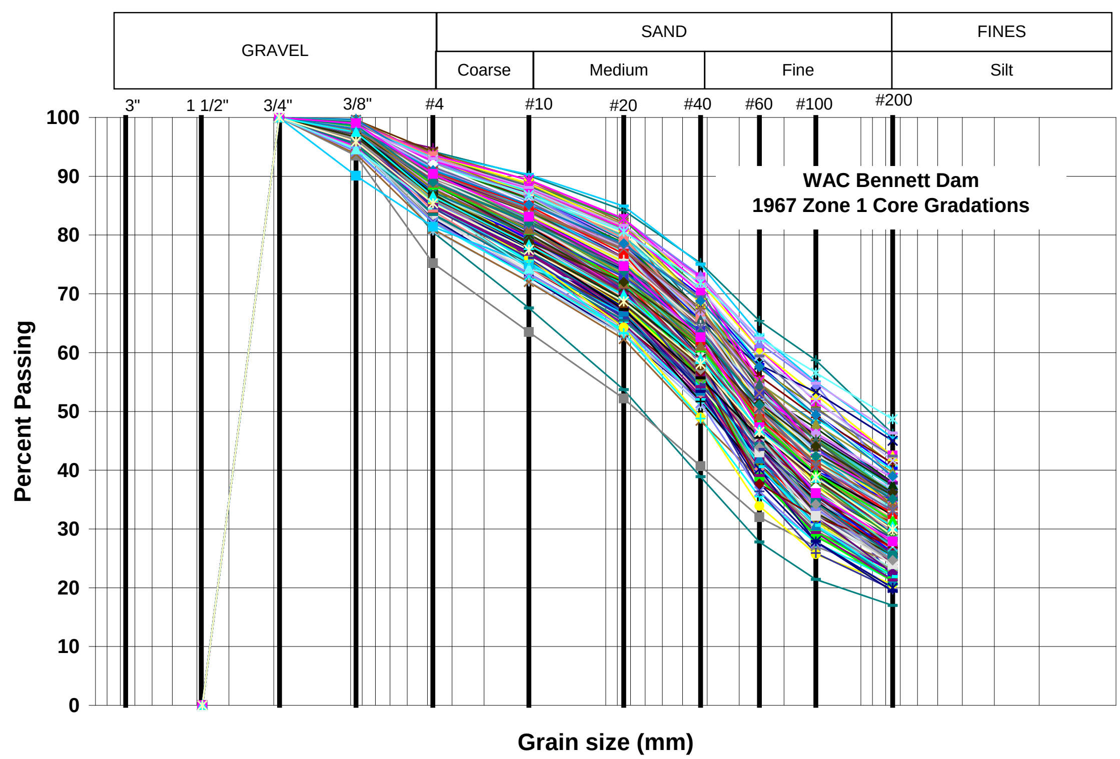
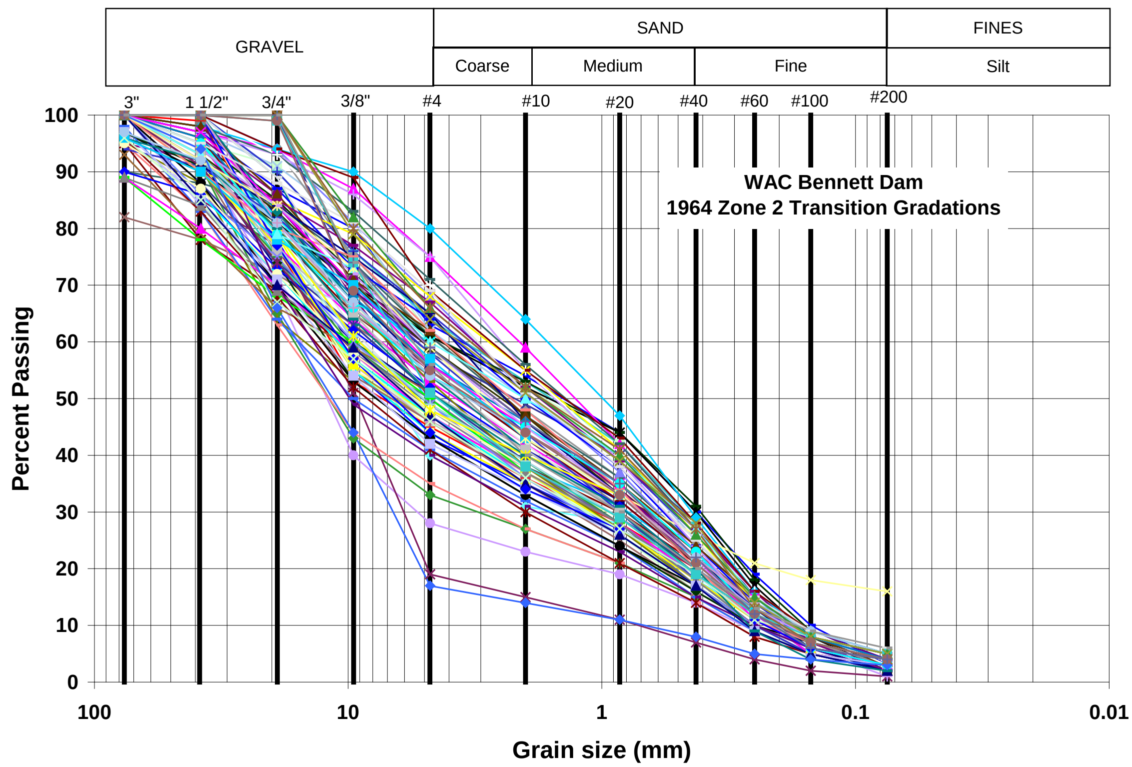


Figure F5b: Gradations of Zone 1 Core from construction records 1966, 1967



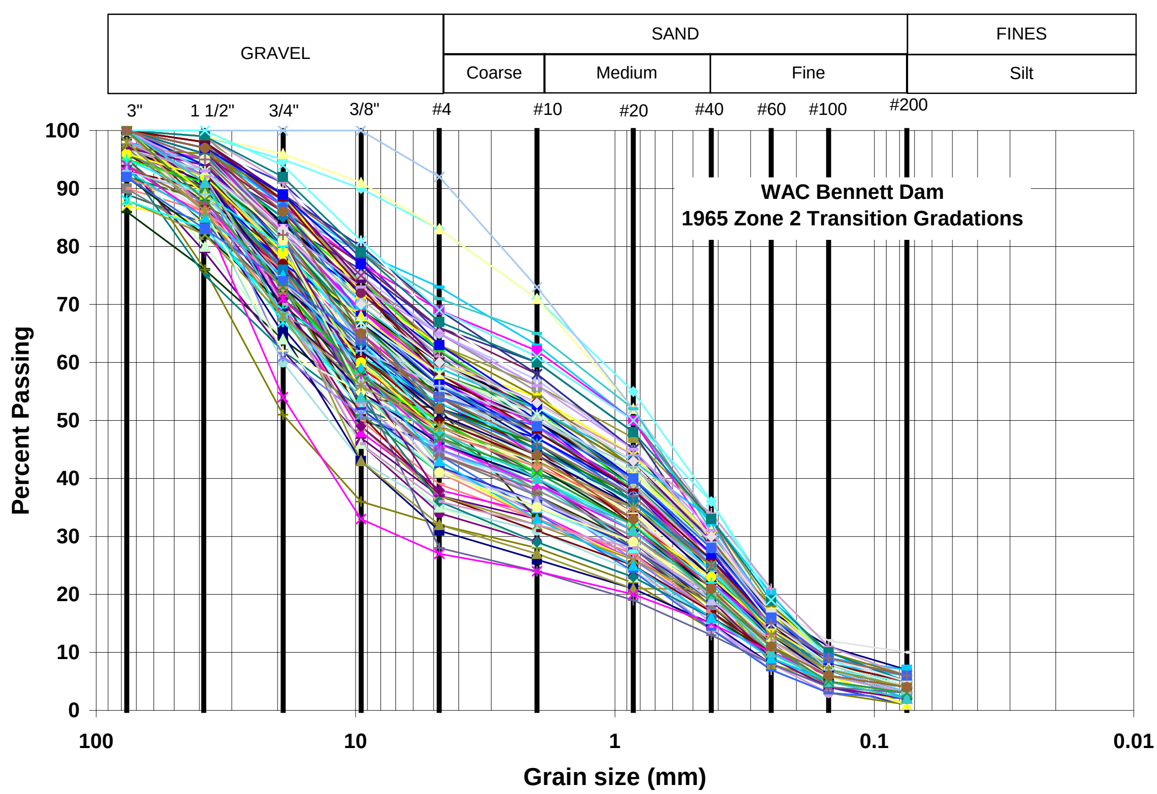
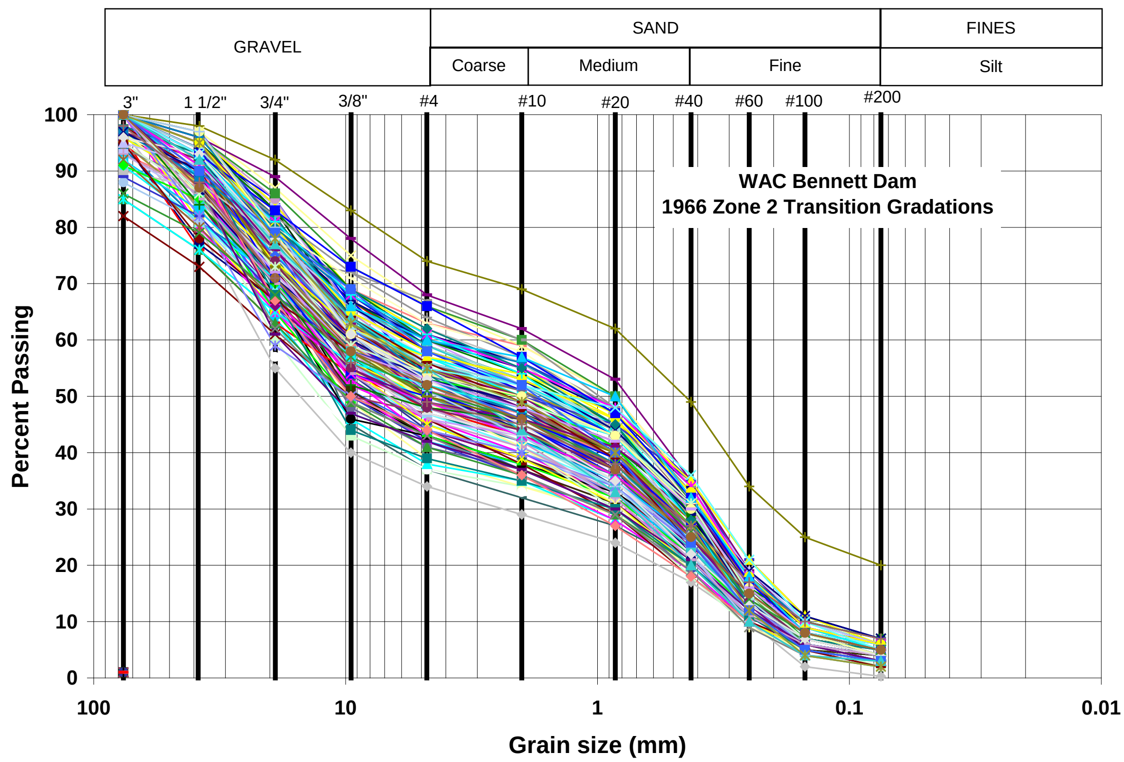


Figure F6a: Gradations of Zone 2 Transition from construction records 1964, 1965



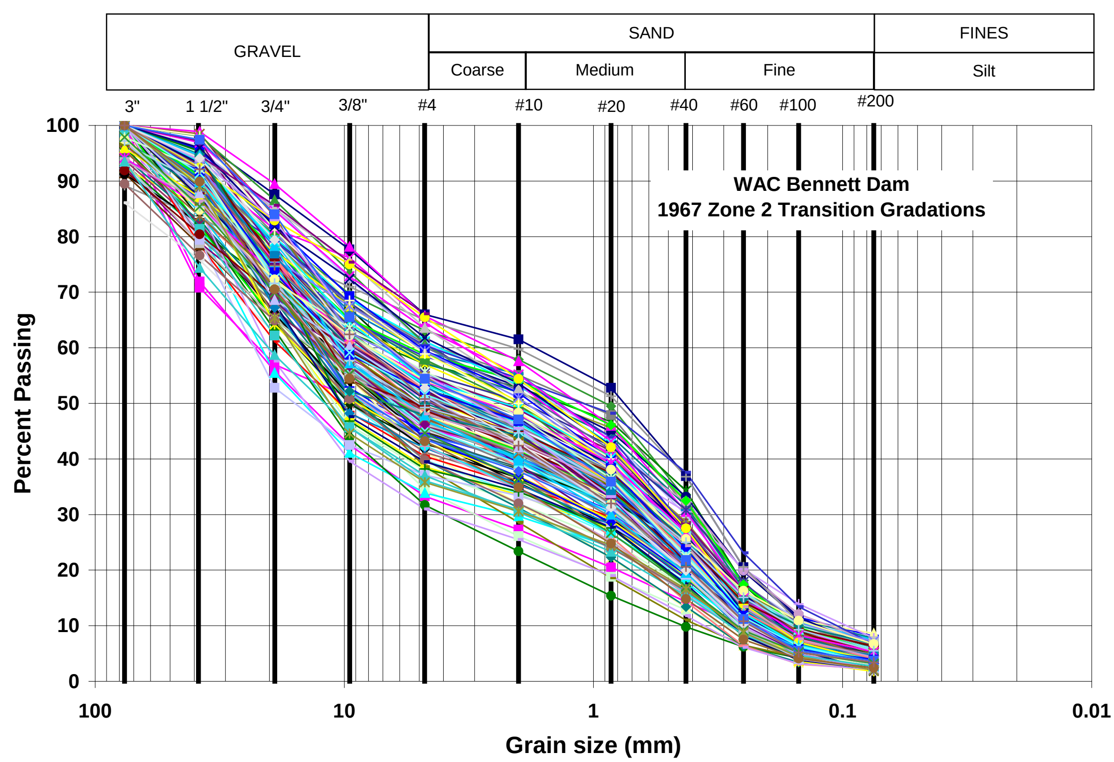
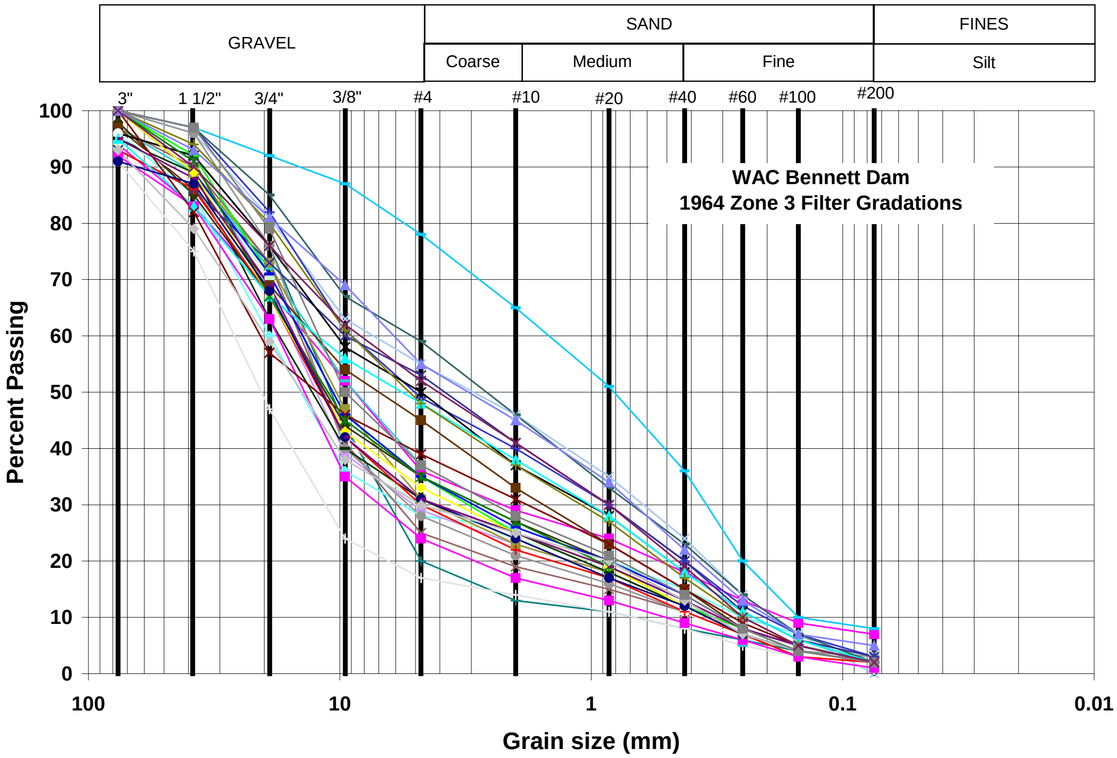


Figure F6b: Gradations of Zone 2 Transition from construction records 1966, 1967



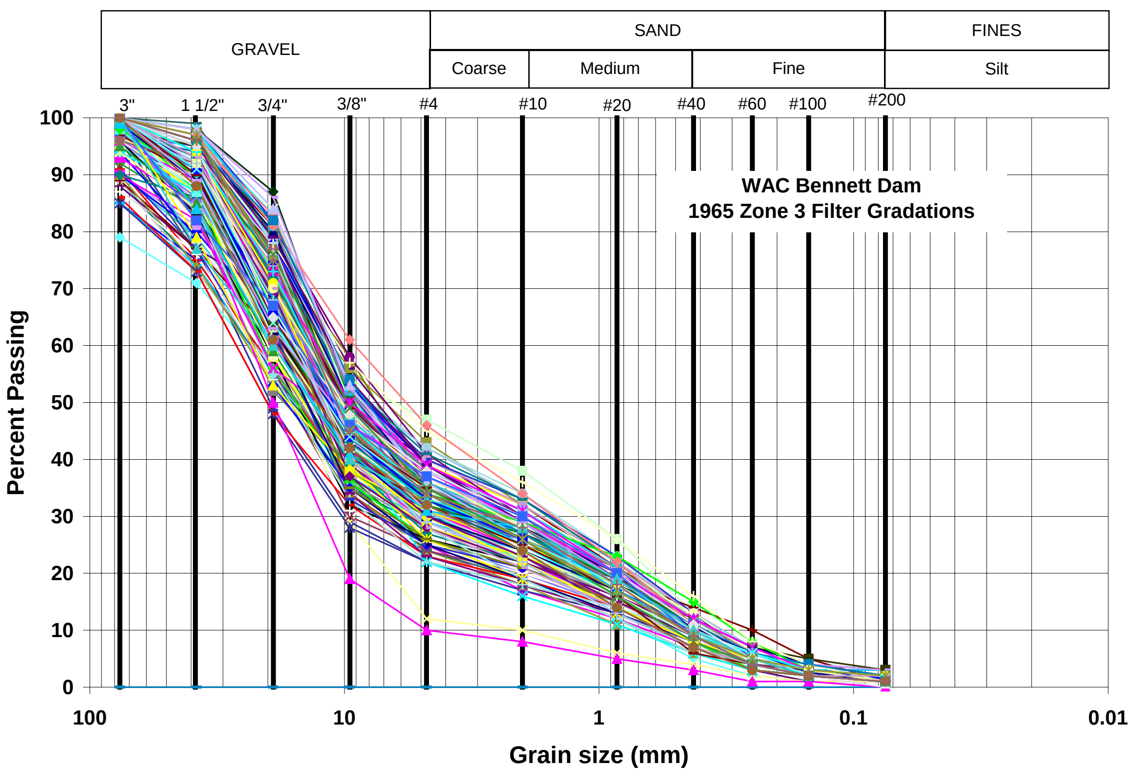
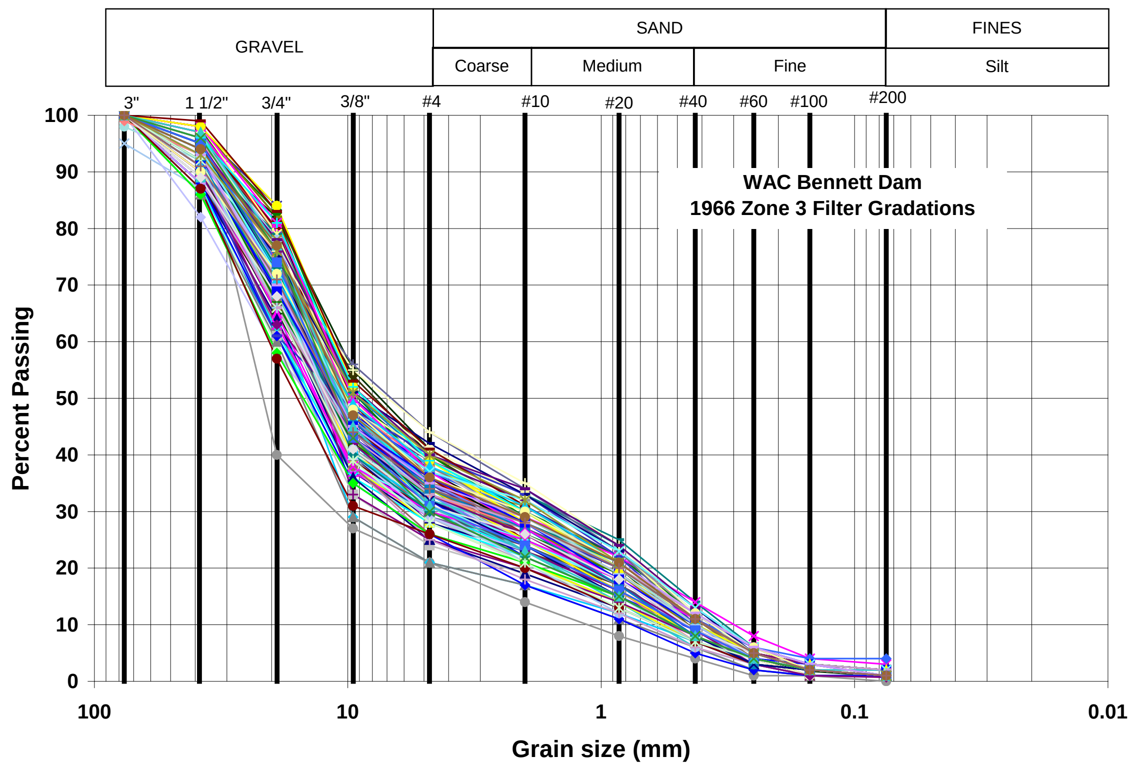


Figure F7a: Gradations of Zone 3 Filter from construction records 1964, 1965



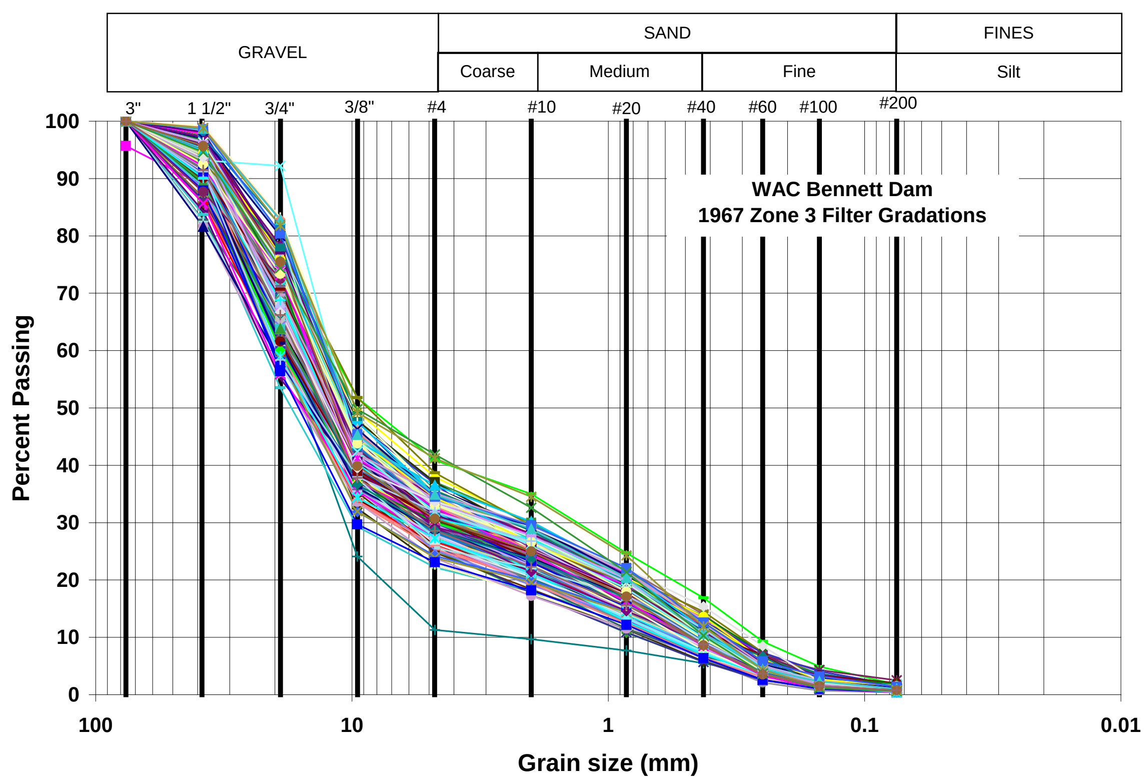
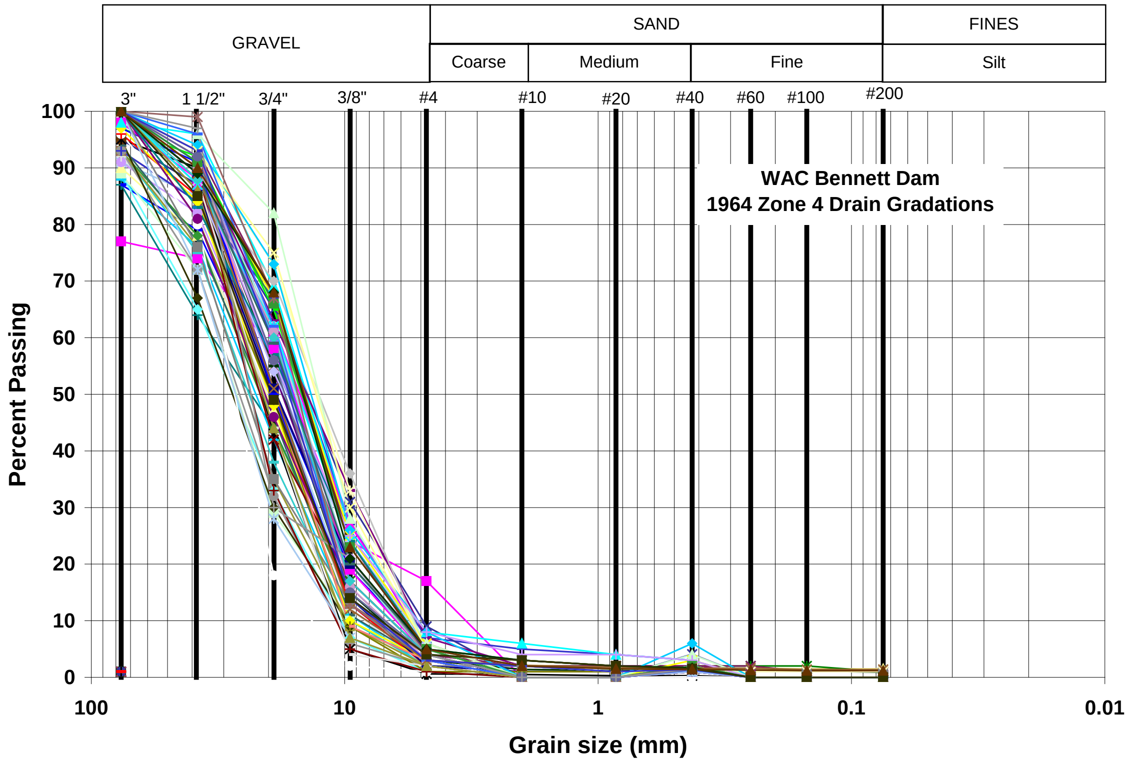


Figure F7b: Gradations of Zone 3 Filter from construction records 1966, 1967



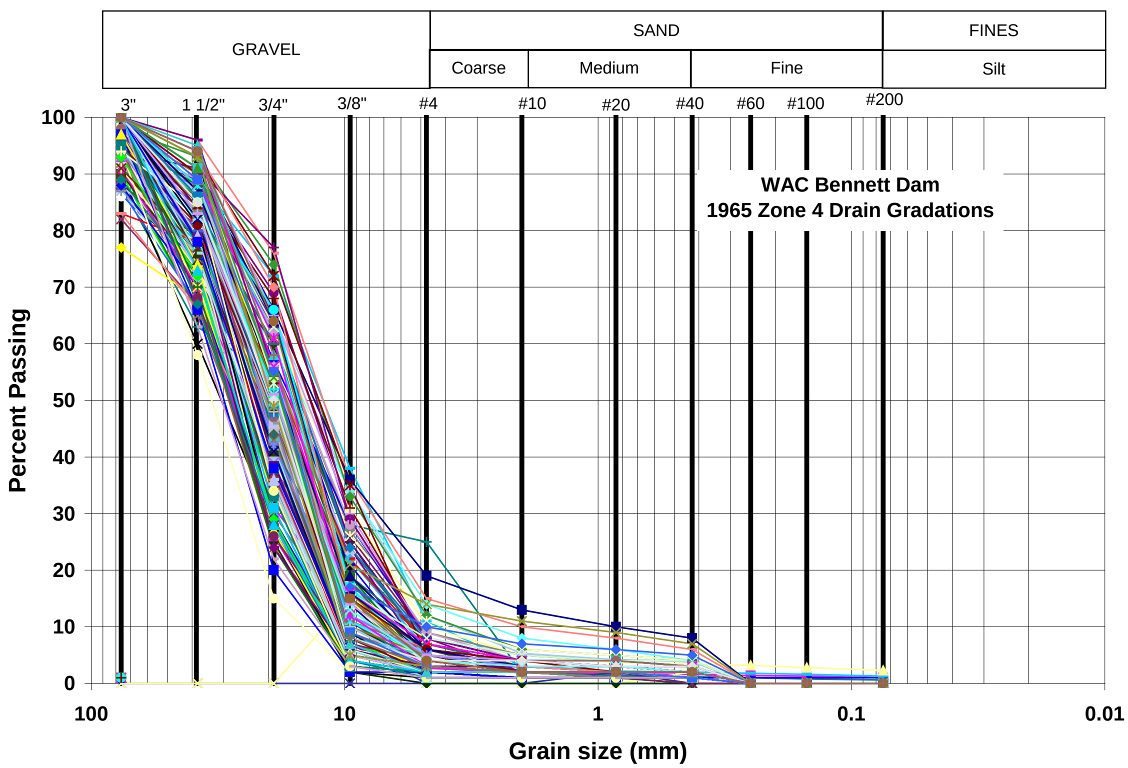
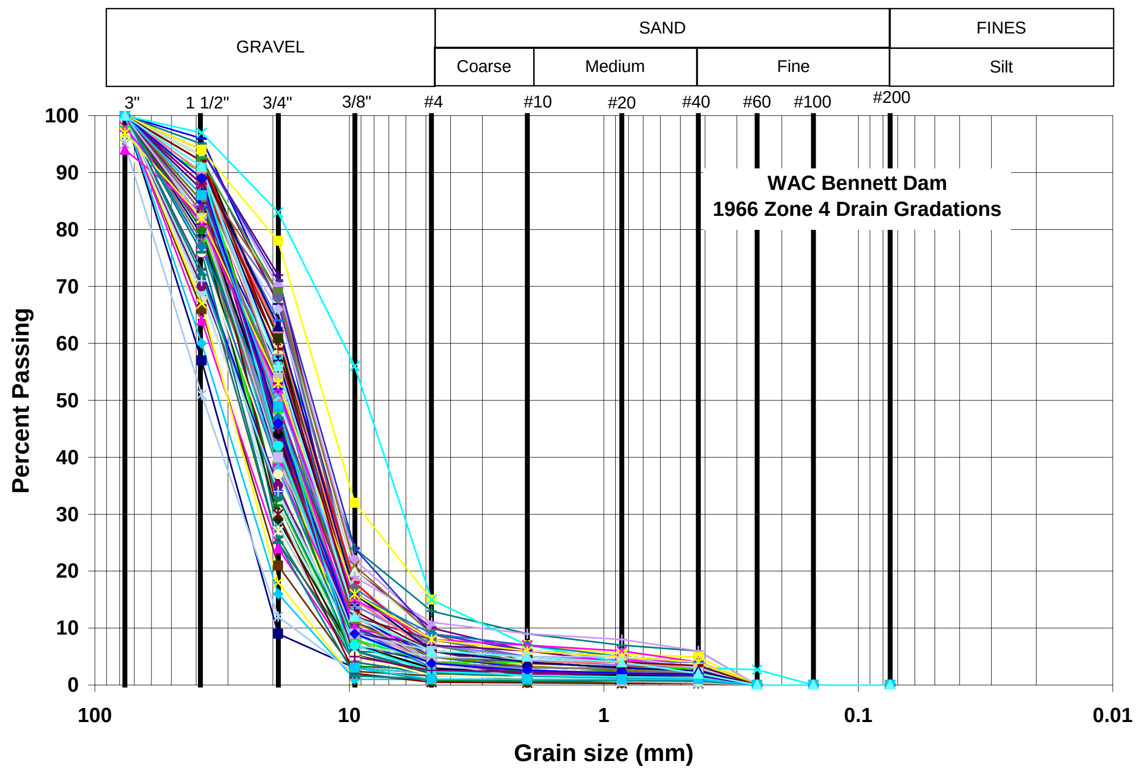


Figure F8a: Gradations of Zone 4 Drain from construction records 1964, 1965



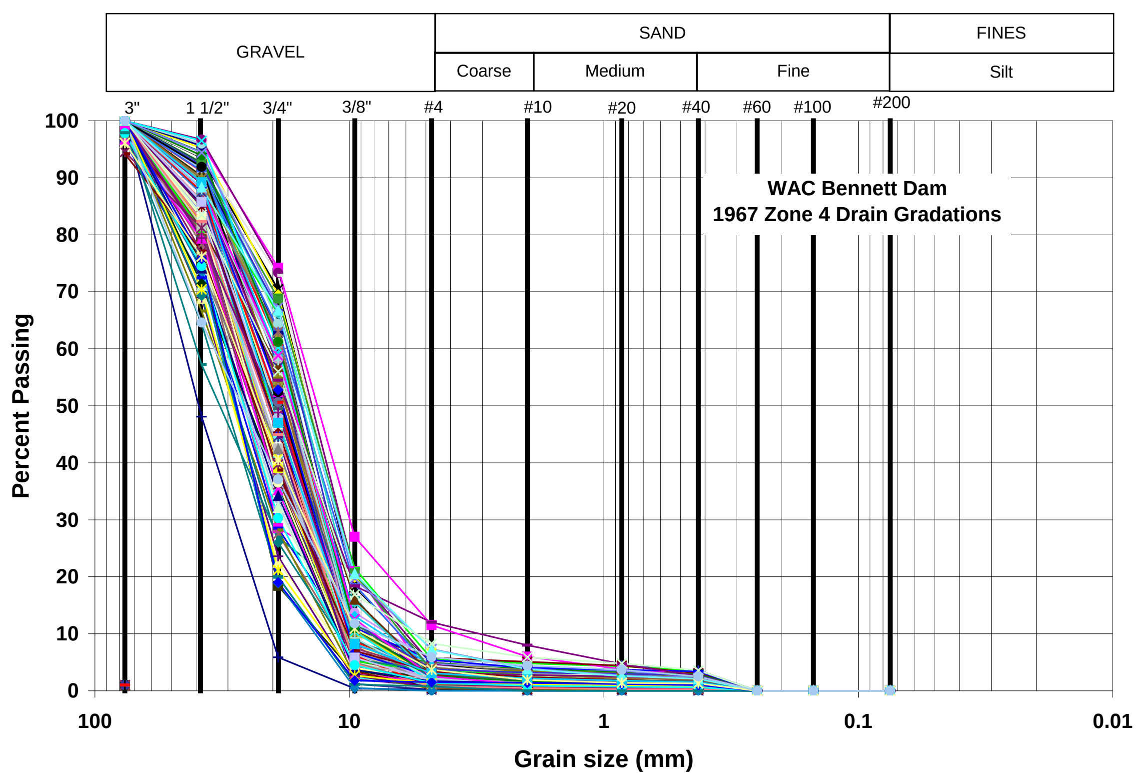


Figure F8b: Gradations of Zone 4 Drain from construction records 1966, 1967

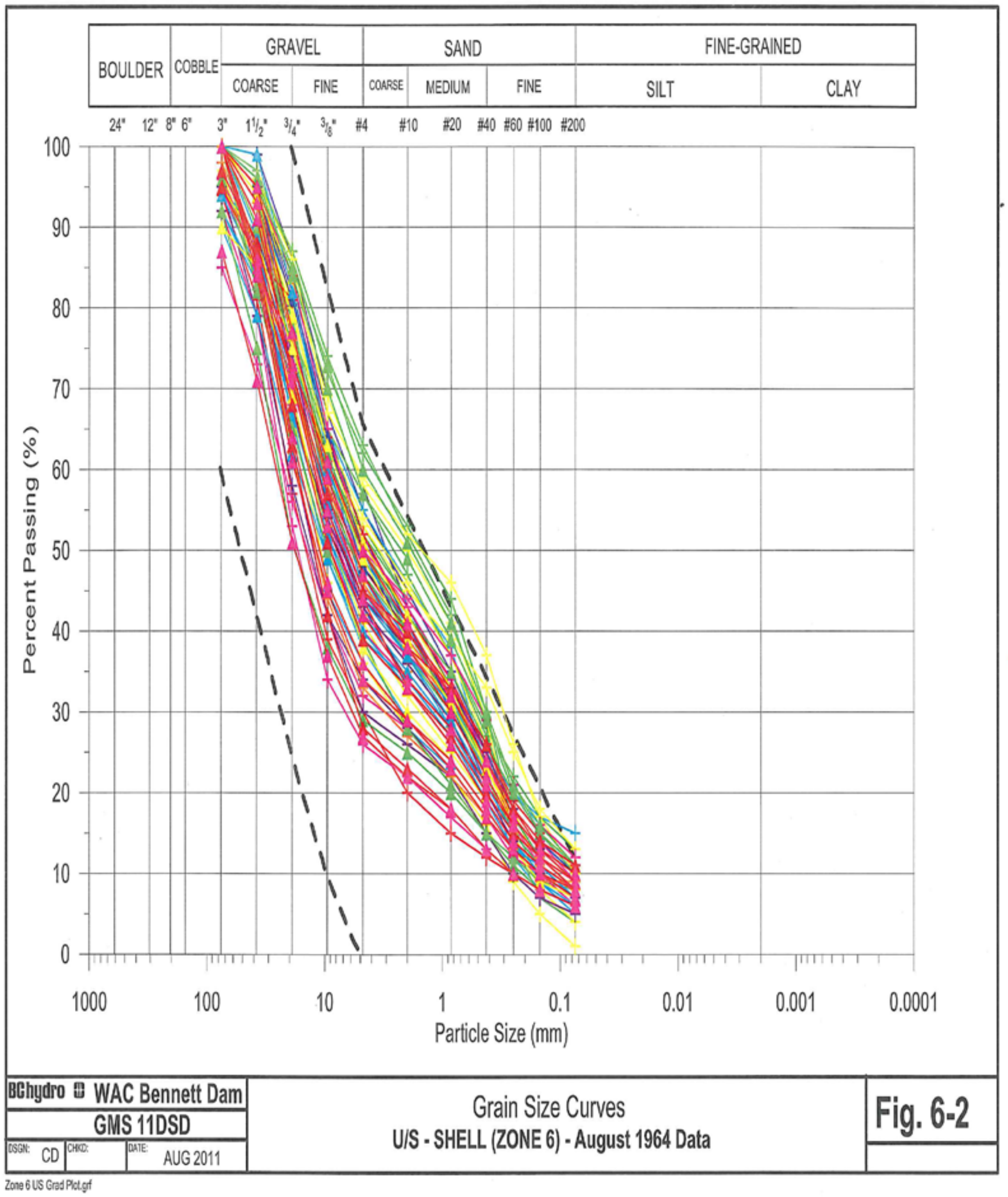


Figure F9a: Gradations of Zone 6 Random Shell from 1964 construction records

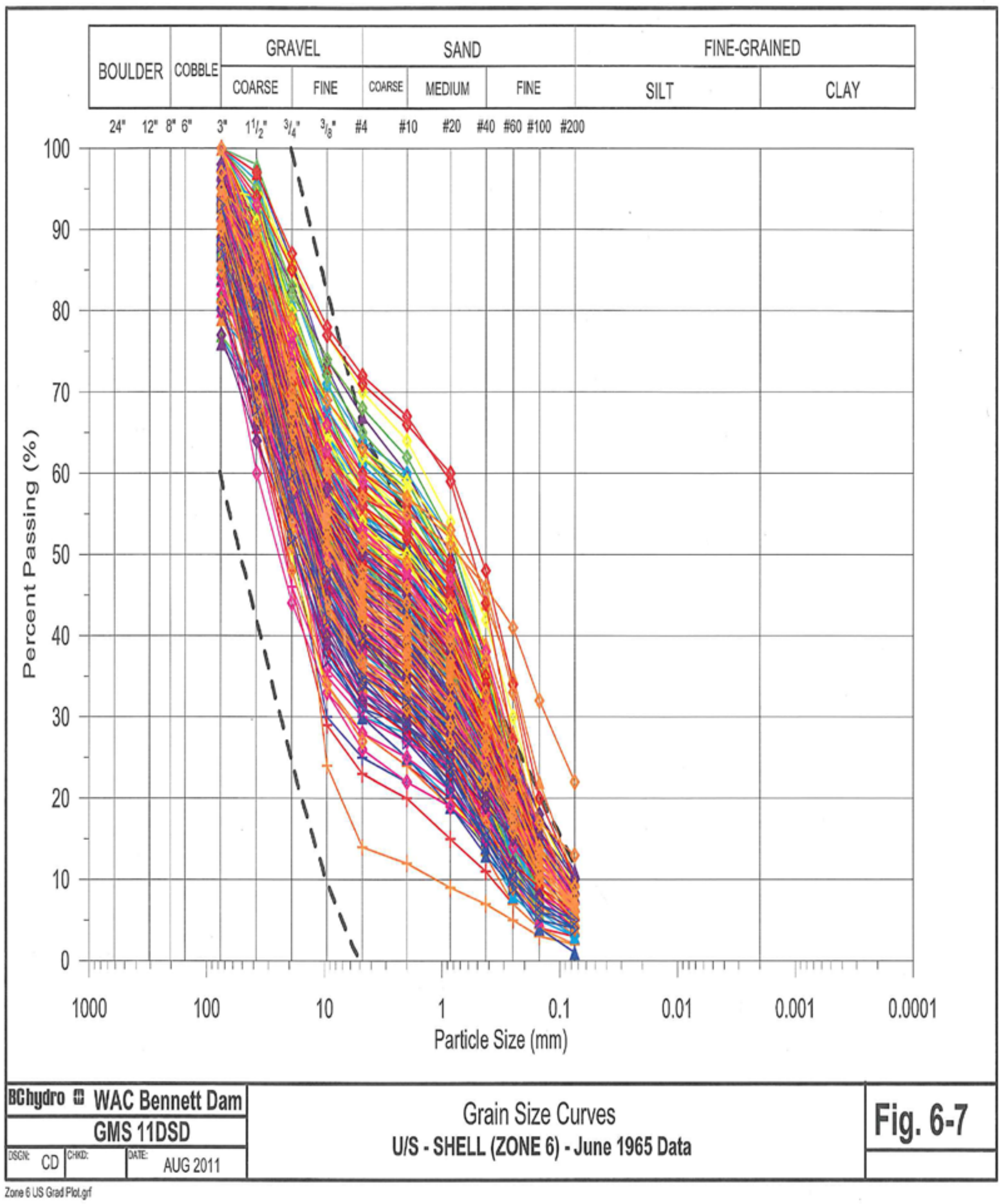


Figure F9b: Gradations of Zone 6 Random Shell from 1965 construction records

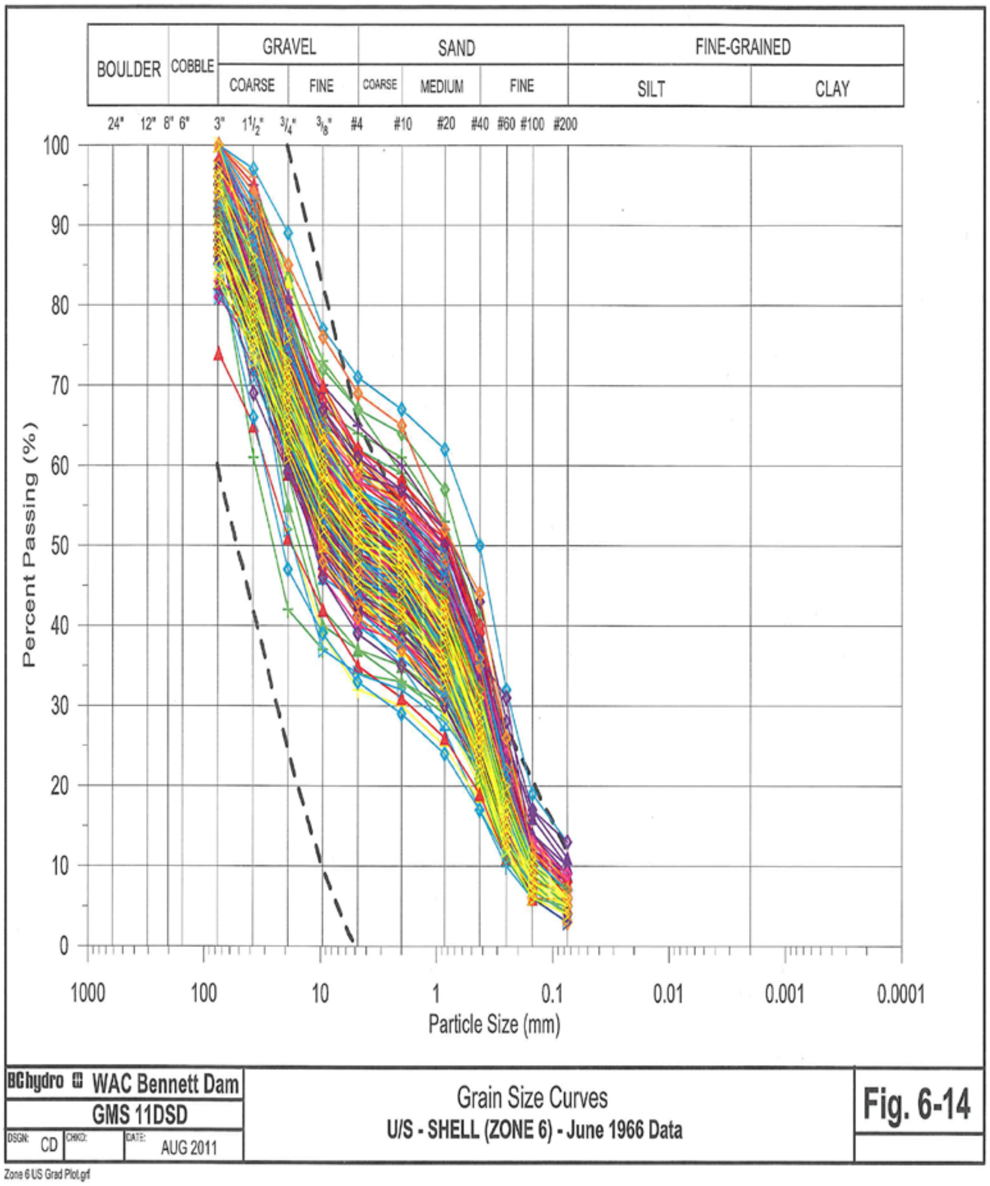


Figure F9c: Gradations of Zone 6 Random Shell from 1966 construction records

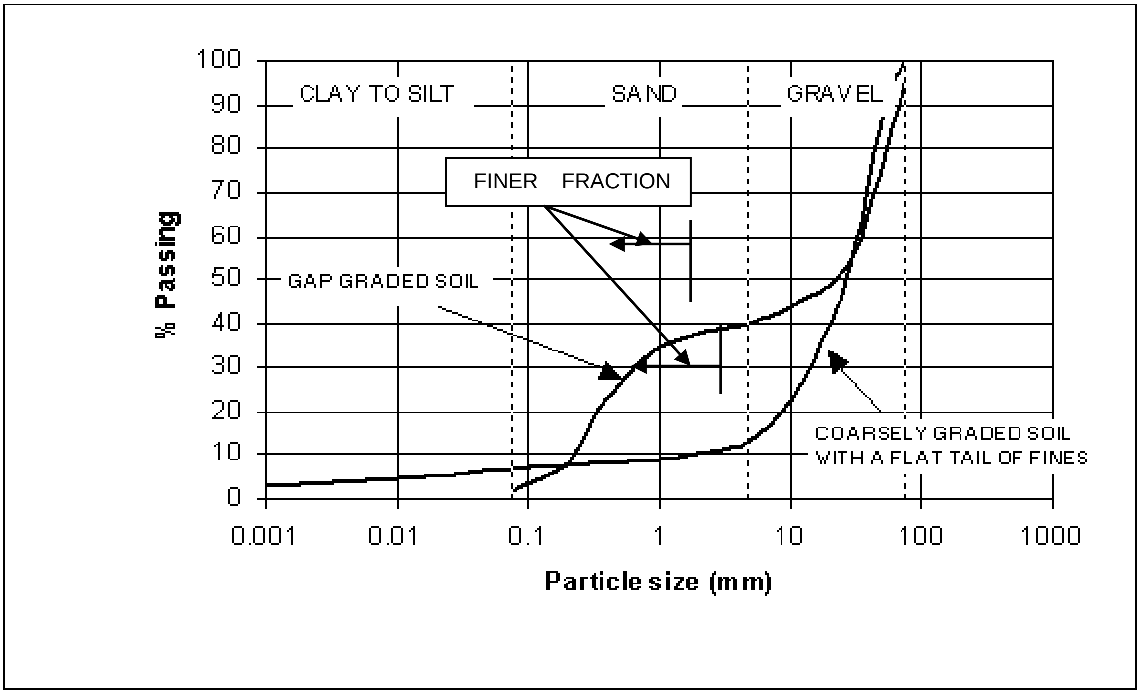


Figure F10: Description of "finer" and "coarser" fractions in potentially internally unstable soils. (ICOLD 2011)

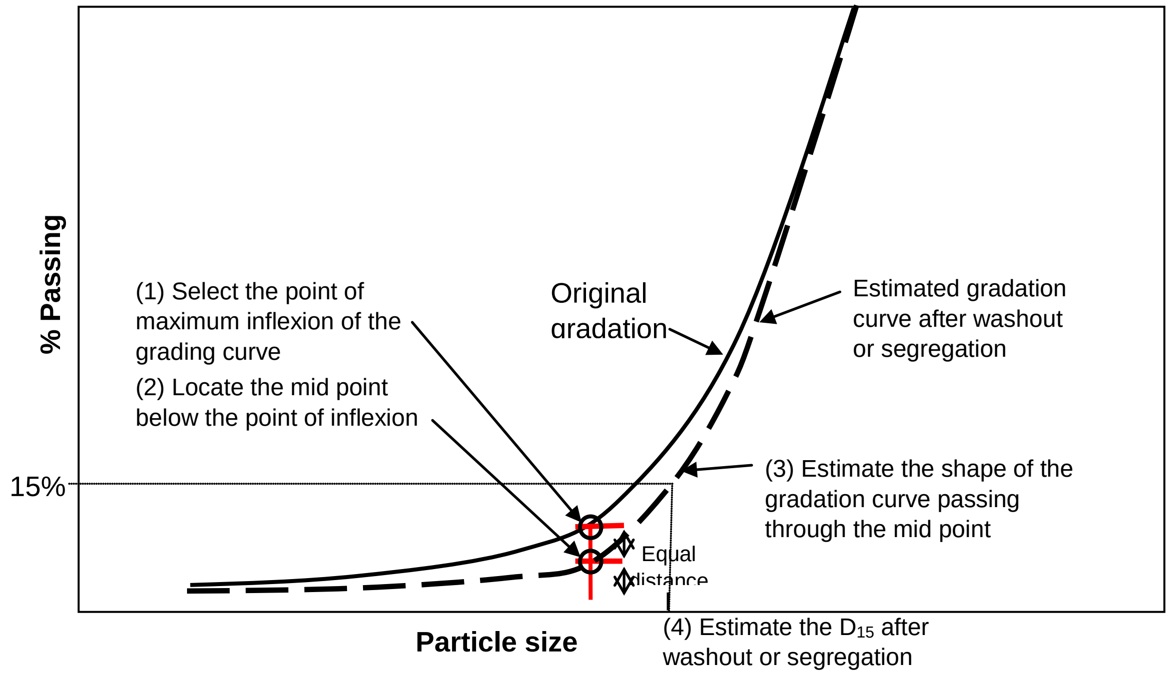


Figure F11: Approximate method for estimating DF15 after washout of the erodible fraction from a suffusive soil or for soils susceptible to segregation (Fell et al 2008)

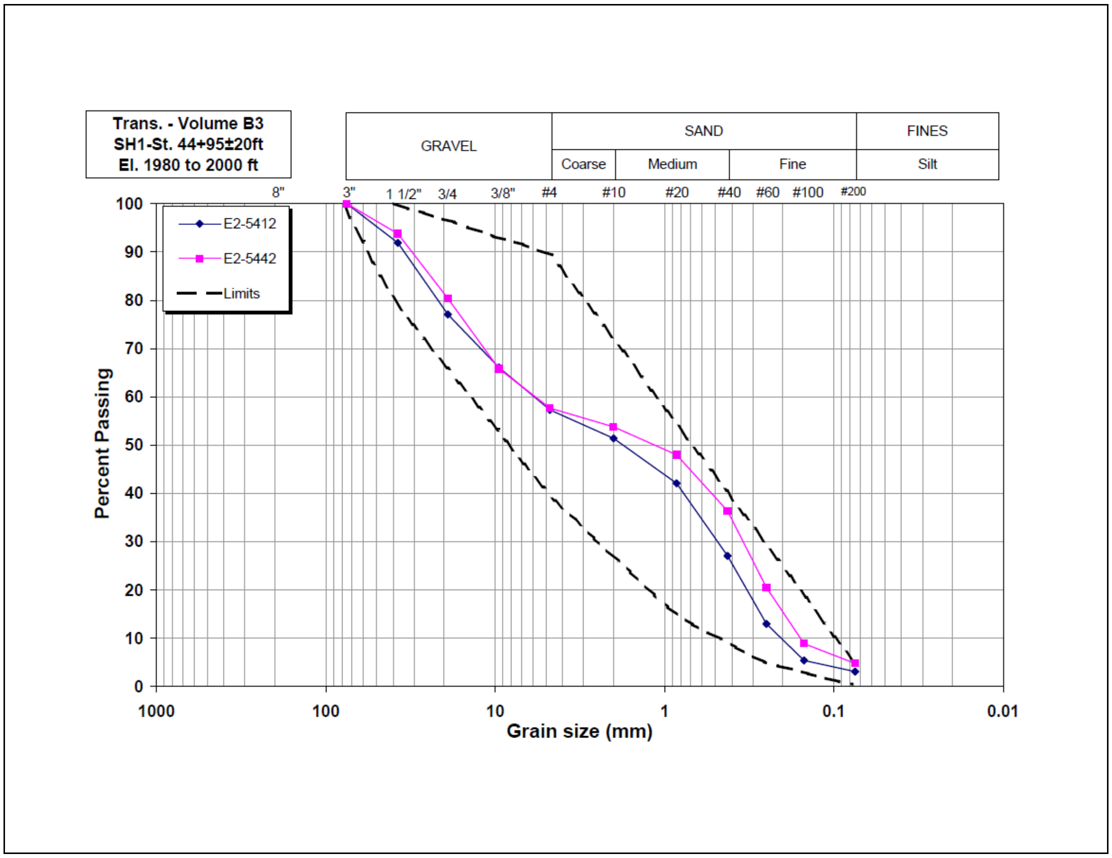


Figure F12: Method for allowing for the effects of segregation of the Transition

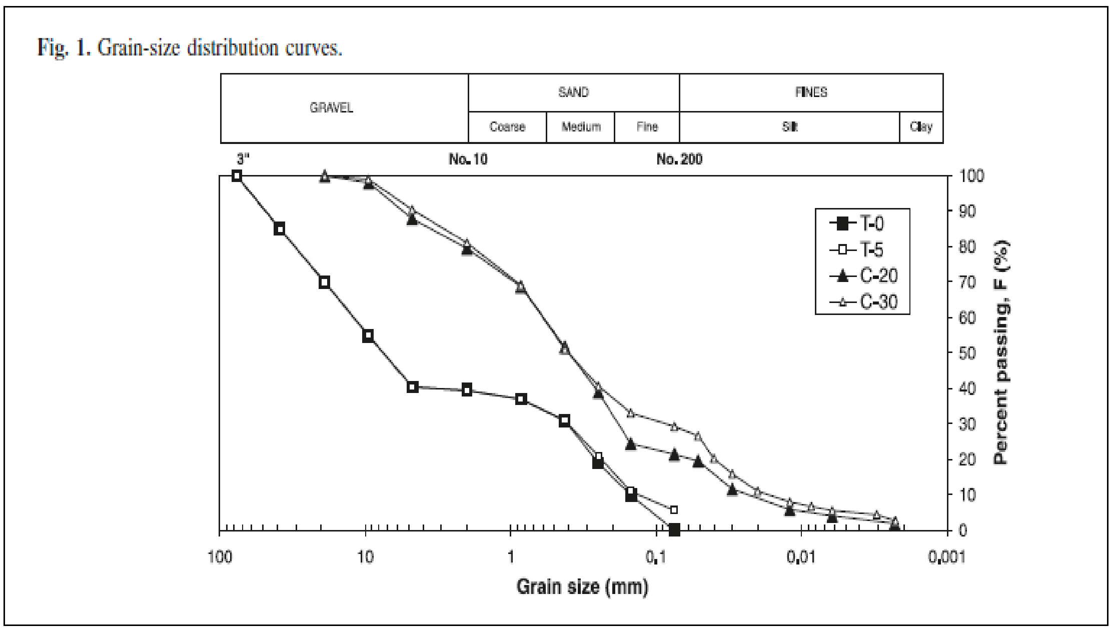


Figure F13: Gradation of samples tested by University British Columbia. Samples T-0 and T-5 represent the Transition, C-20 and C-30 the Core.

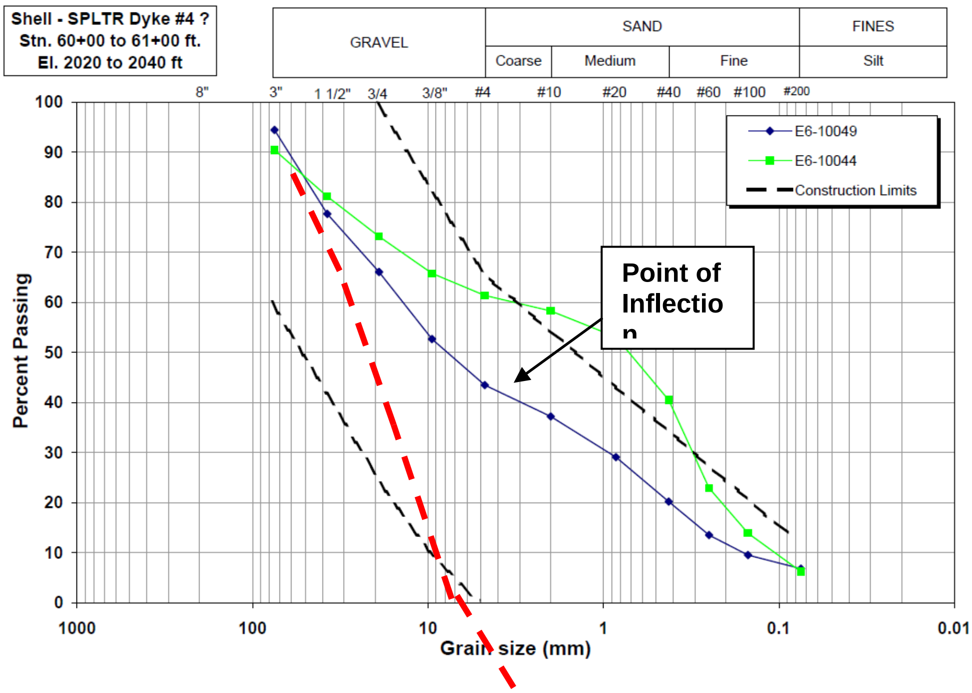


Figure F14: Gradations of Zone 6 Random Shell at Splitter Dike No 4 from construction records

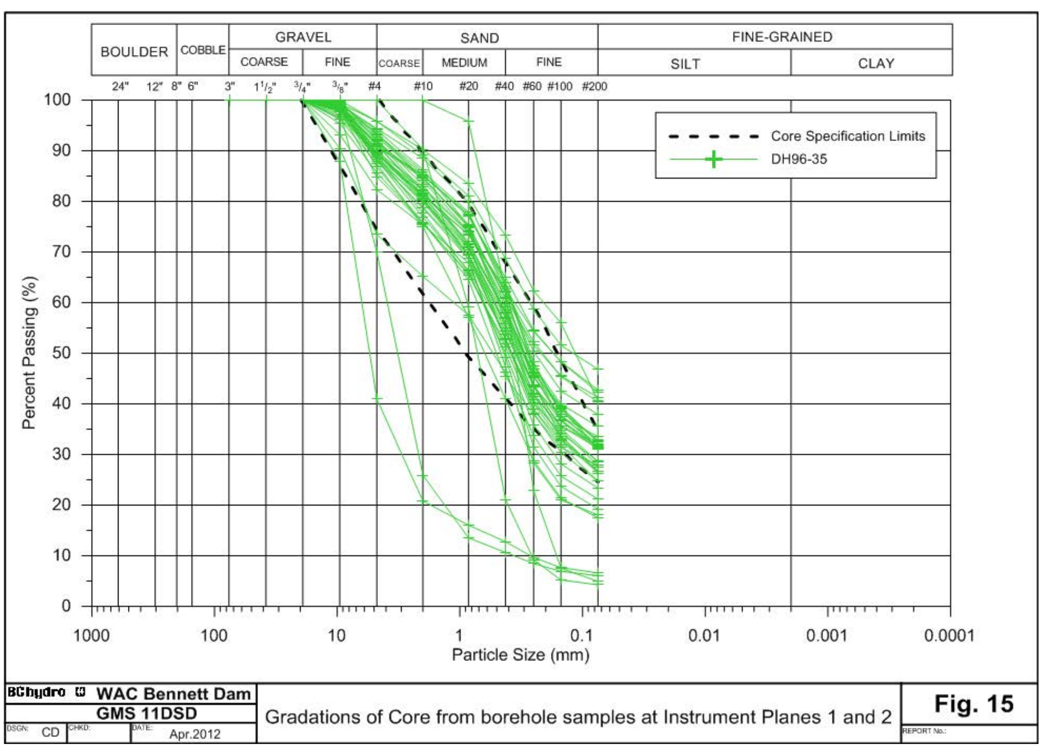


Figure F15: Gradations of Core from borehole samples at Instrument Planes 1 and 2

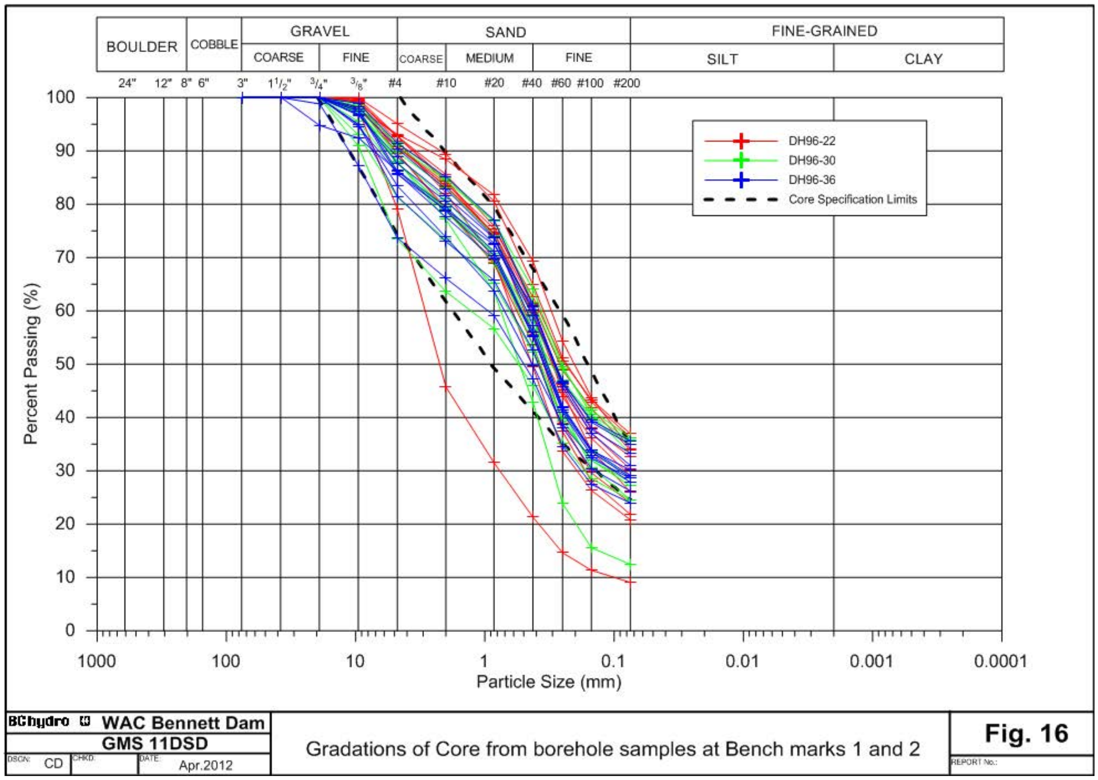


Figure F16: Gradations of Core from borehole samples at Bench marks 1 and 2

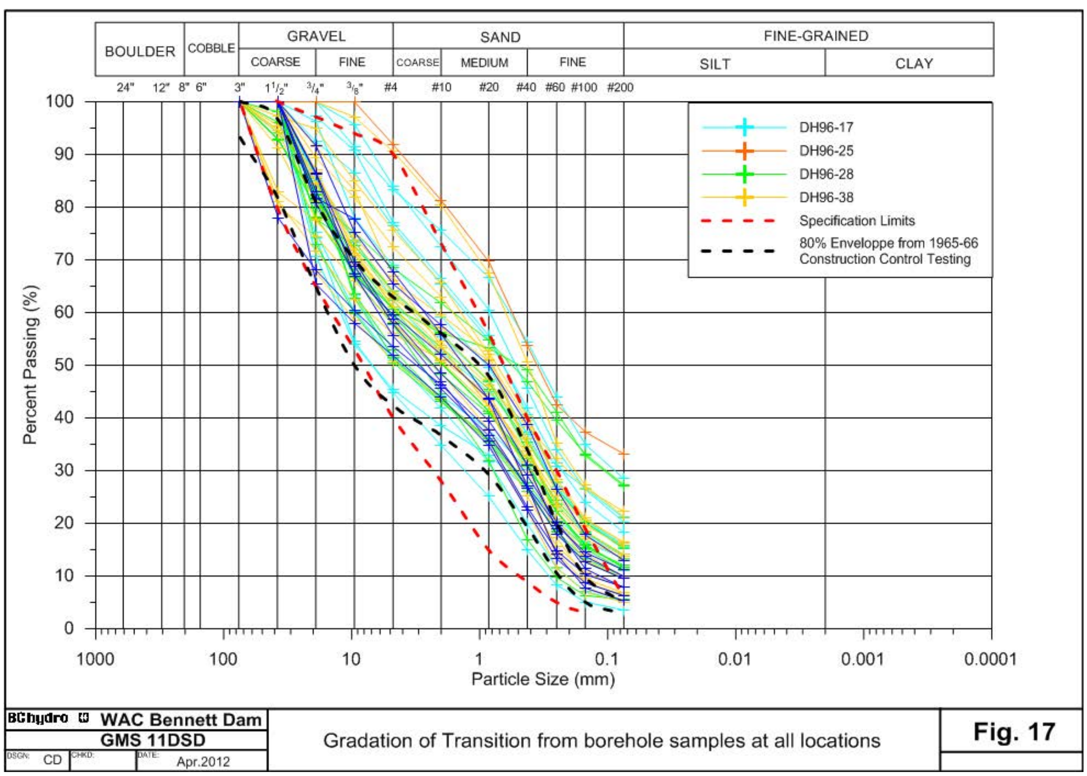


Figure F17: Gradation of Transition from borehole samples at all locations

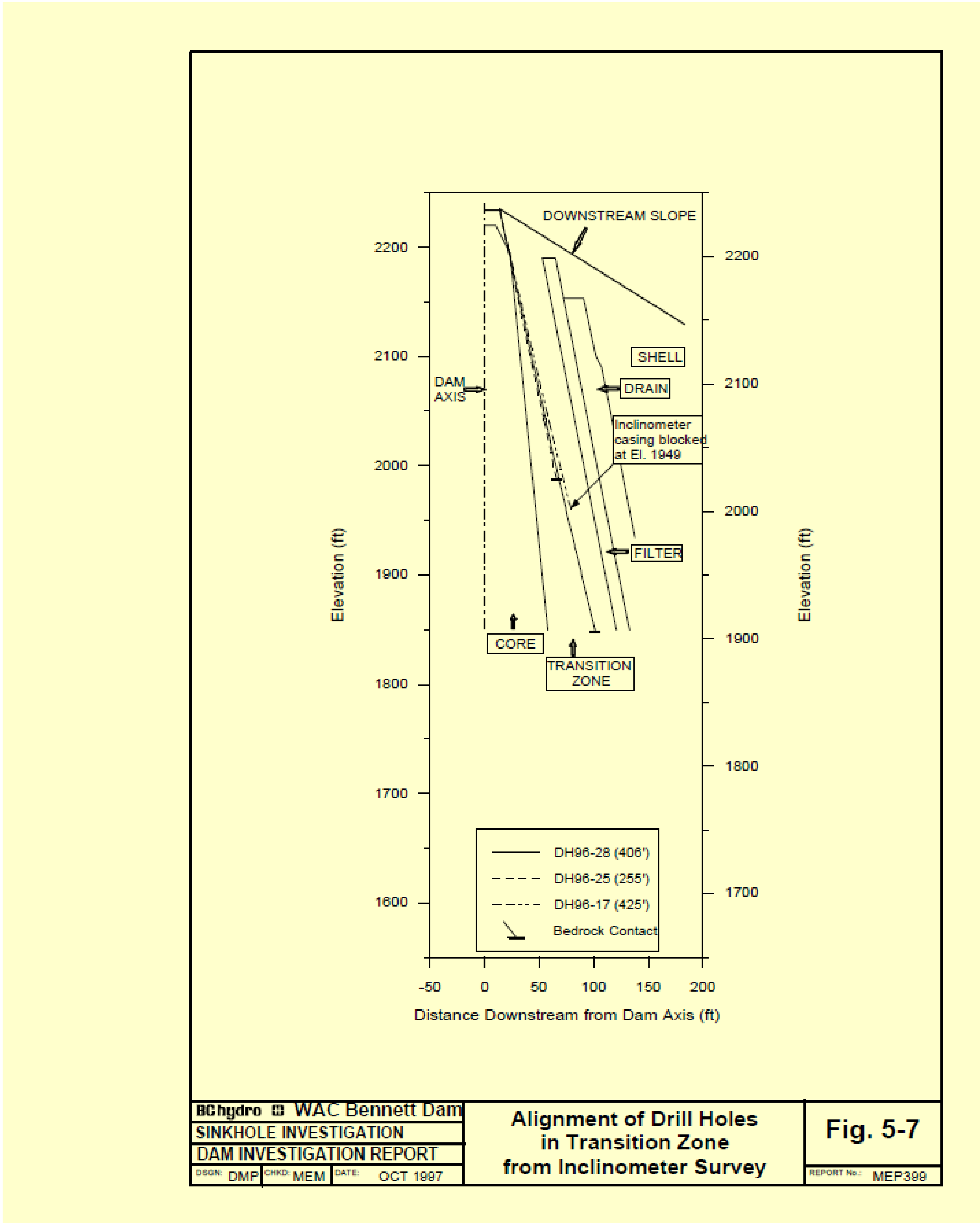


Figure F18: Alignment of drill holes in the Transition in 1996 investigations

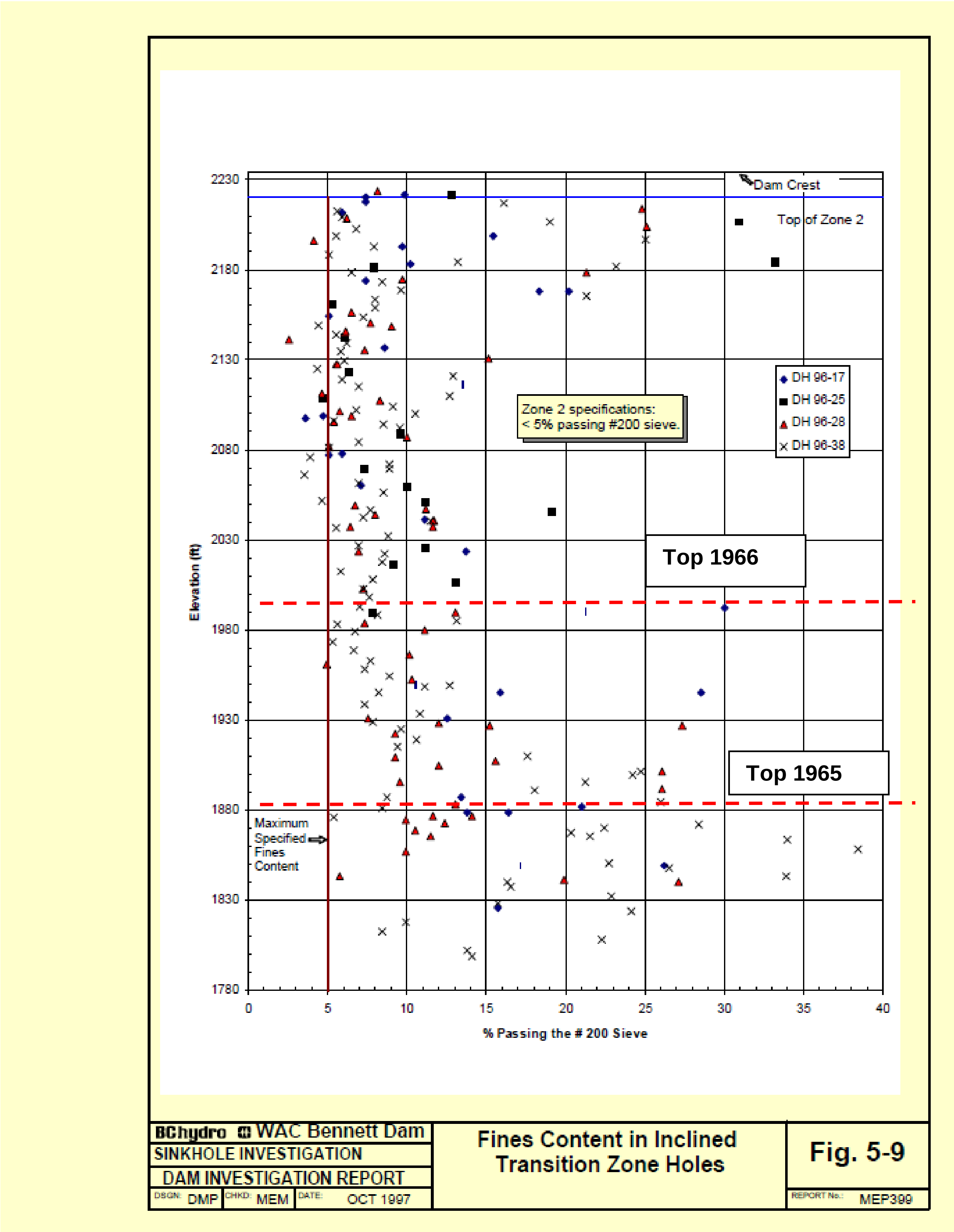


Figure F19: Fines content (% passing 0.075mm sieve) in samples taken from 1996 boreholes in the Transition. Also shown are elevations of top of 1965 and 1966 construction.

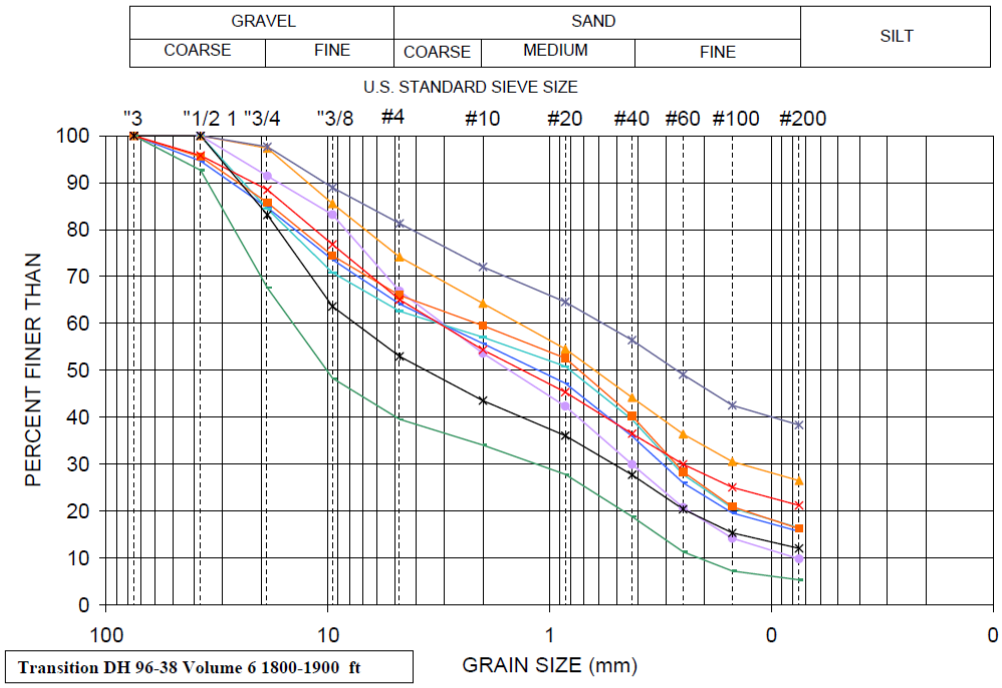


Figure F20: Gradations of Transition from borehole DH 96-38, 1800ft to 1900ft

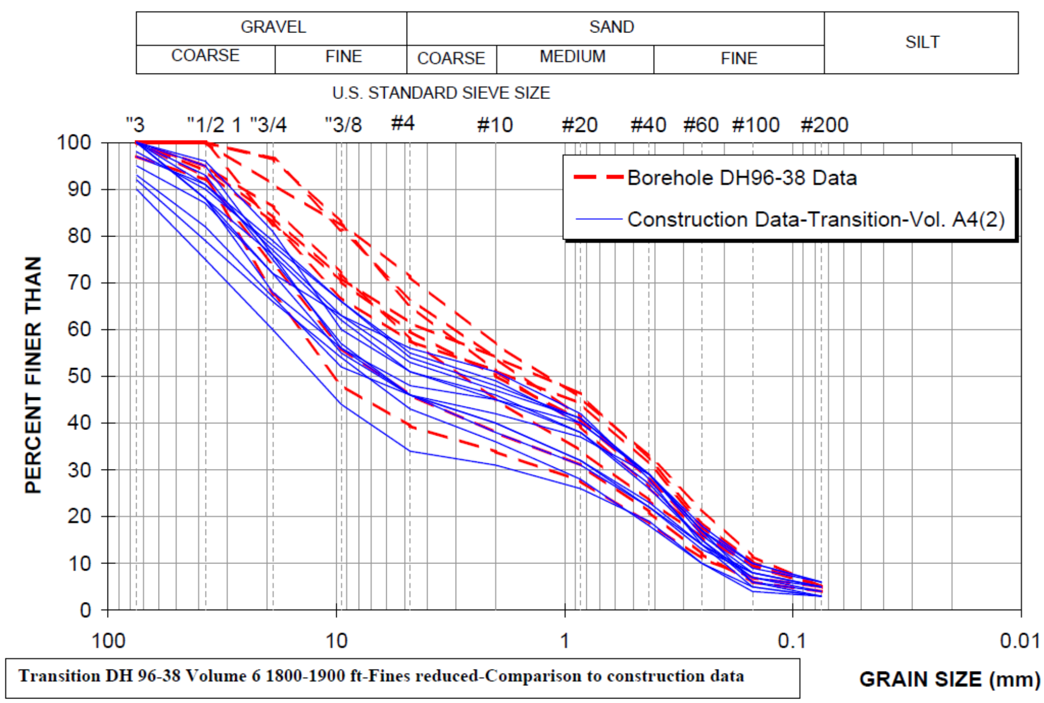


Figure F21: Gradations of Transition from borehole DH-96-38 regraded to 5% fines, and as constructed gradations from adjacent to the borehole location.

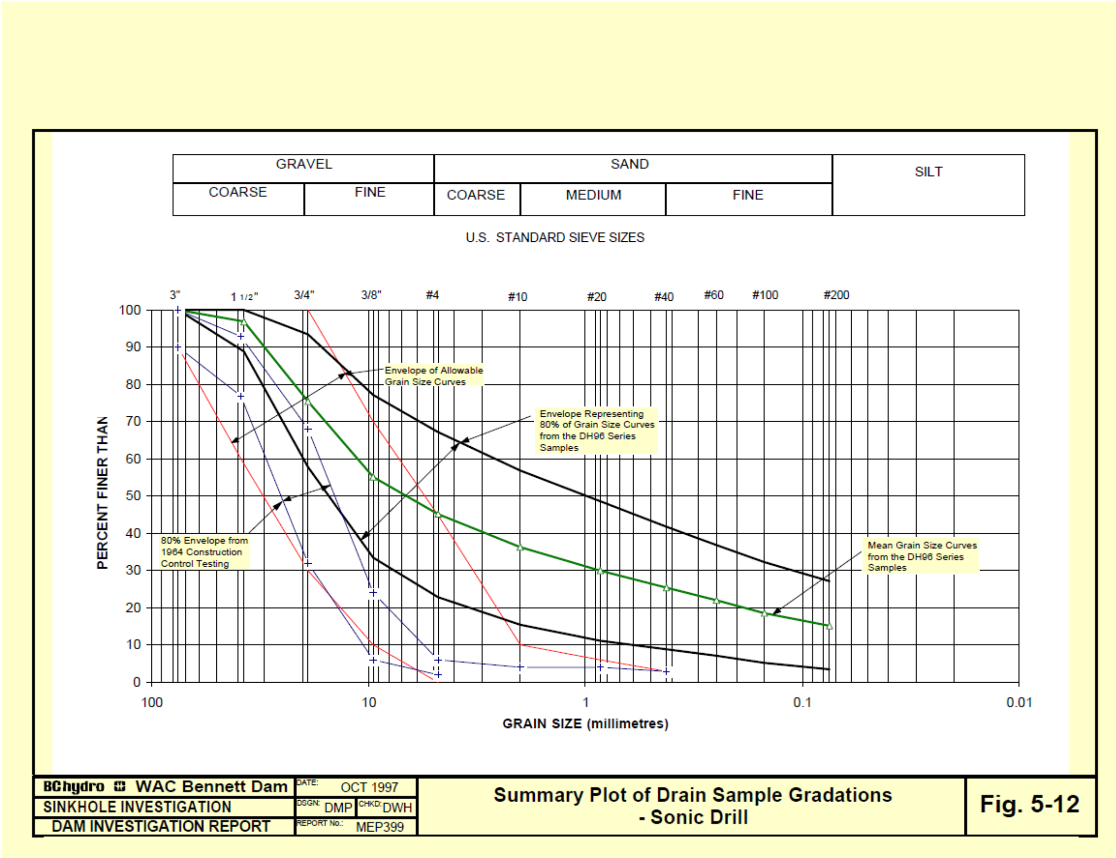


Figure F22: Drain sample gradations from samples taken in 1996 investigations compared to as placed gradations.

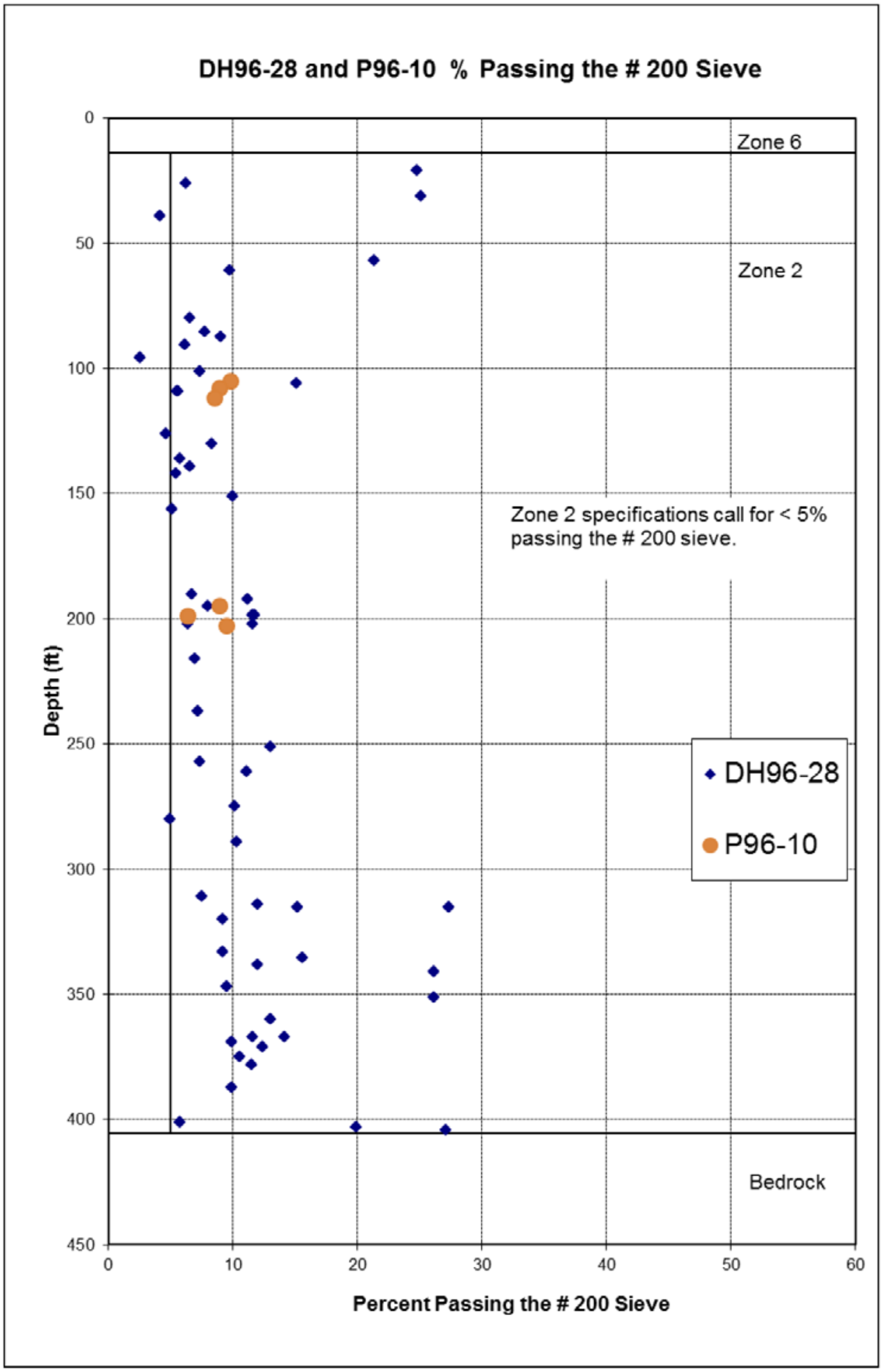


Figure F23: Plot of % Fines passing 0.075mm sieve versus depth for adjacent Boreholes drilled using Sonic and Barber Drills.

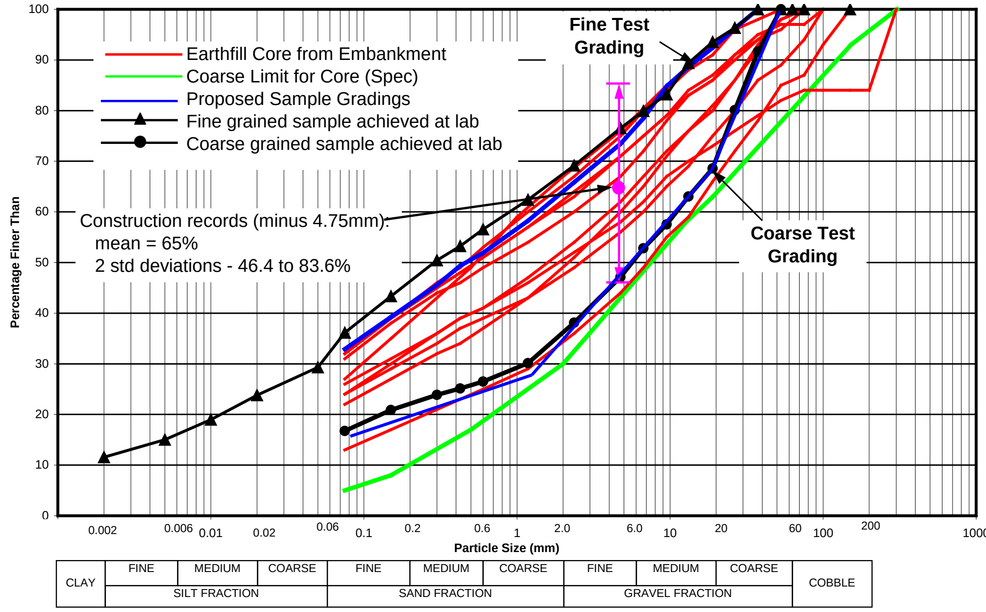


Figure F24: Gradation of Australian soil tested in global backward erosion tests. (NOTE: THESE DATA NOT TO BE REPRODUCED)

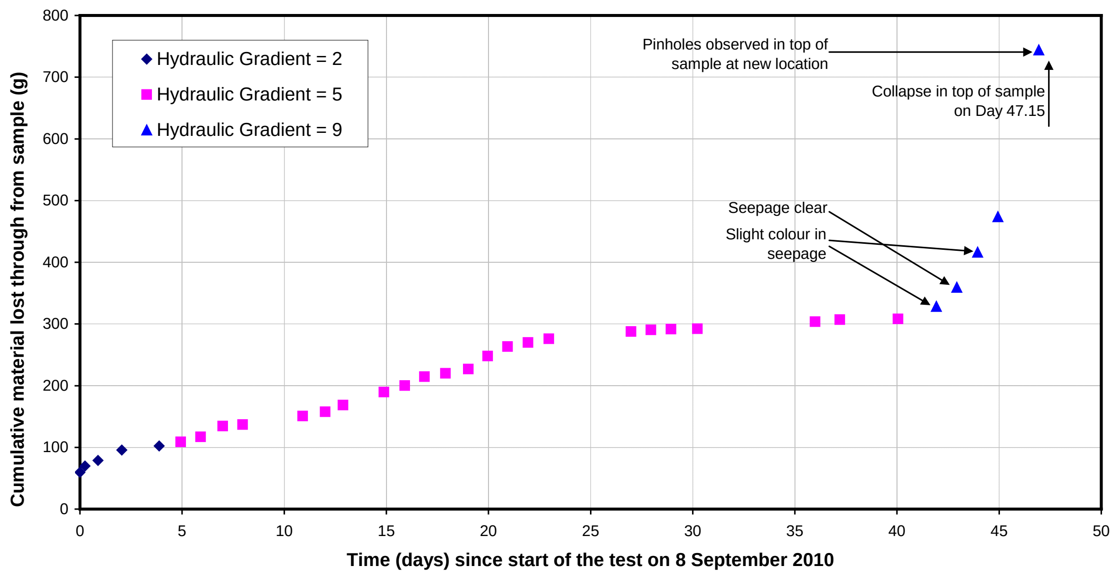


Figure F25: Cumulative erosion versus time for backward erosion test. (NOTE: THESE DATA NOT TO BE REPRODUCED)

Appendix G: Instrument Installations Figures

APPENDIX G

INSTRUMENT INSTALLATIONS FIGURES

[redacted]

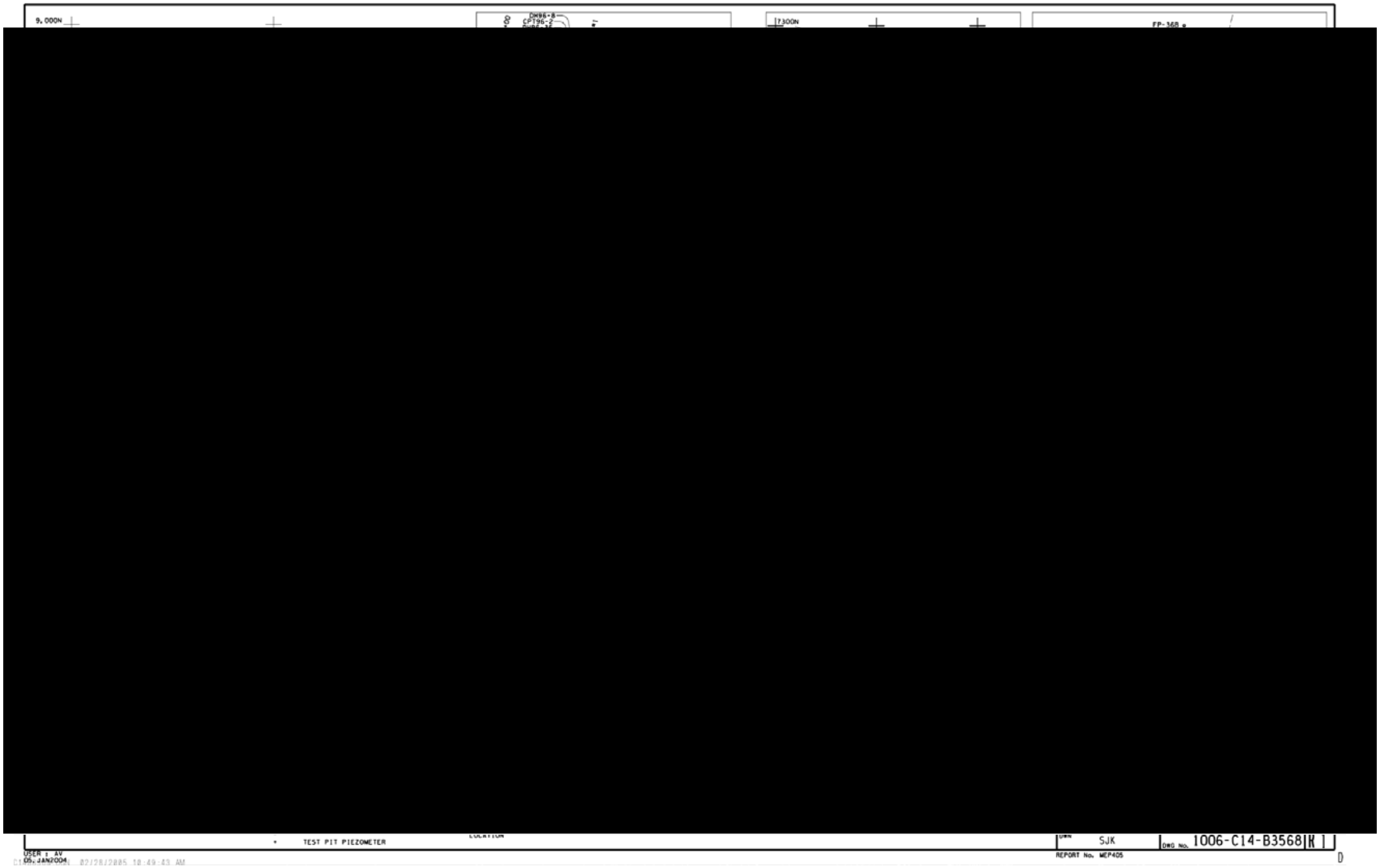


Figure G1: Instrument Location Plan

[redacted]

Figure G2: Location of instruments in Instrument Plane 1 with riser highlighted

[redacted]

Figure G3: Location of instruments Instrument Plane 2 with riser highlighted

[redacted]

Figure G4: Cross Hole seismic survey data for Plane 10, OW2 to DH96-35

[redacted]

Figure G5: Cross Hole seismic survey data for Plane 31, CX 1 to OW2

[redacted]

Figure G6: Cross Hole seismic survey data for Plane 19, OW4 to DH96-14

[redacted]

Figure G7: Cross Hole seismic survey data for Plane 25, OW9 to DH96-31

Figure G8: Photograph of frozen Core Zone 1 fill, October 1964.

Appendix H: Mechanism of Formation of Sinkholes at Benchmarks 1 and 2

APPENDIX H

MECHANISM OF FOUNDATION OF SINKHOLES AT BENCHMARKS 1 AND 2

CONTENTS

LIST OF TABLES AND FIGURES

APPENDIX H

MECHANISM OF FOUNDATION OF SINKHOLES AT BENCHMARKS 1 AND 2

H1 DATA UPON WHICH THE DISCUSSION IS BASED

The information which is available is in the following reports:

MEP 400, Sinkhole Characterization Report Sections 2, 3, and 4.

MEP 400, Sinkhole Characterization Report, Appendix E.

H2 SCOPE OF THE DISCUSSION

This discussion concentrates on the wetting compaction hypothesis. It reviews some of the assumptions made for the analyses done for the reports and looks at the implications of the assumptions on the likelihood that the sinkhole formation can be explained by the wetting compaction hypothesis alone or whether some internal erosion has occurred.

H3 SOME INFORMATION REGARDING THE GEOMETRY OF THE SINKHOLES

H3.1 Sinkhole No 1 at Benchmark 1

Figure H1 shows the evolution of the sinkhole during investigations. At Stage 2 the sinkhole was 21 ft deep and 8 to 10ft diameter.

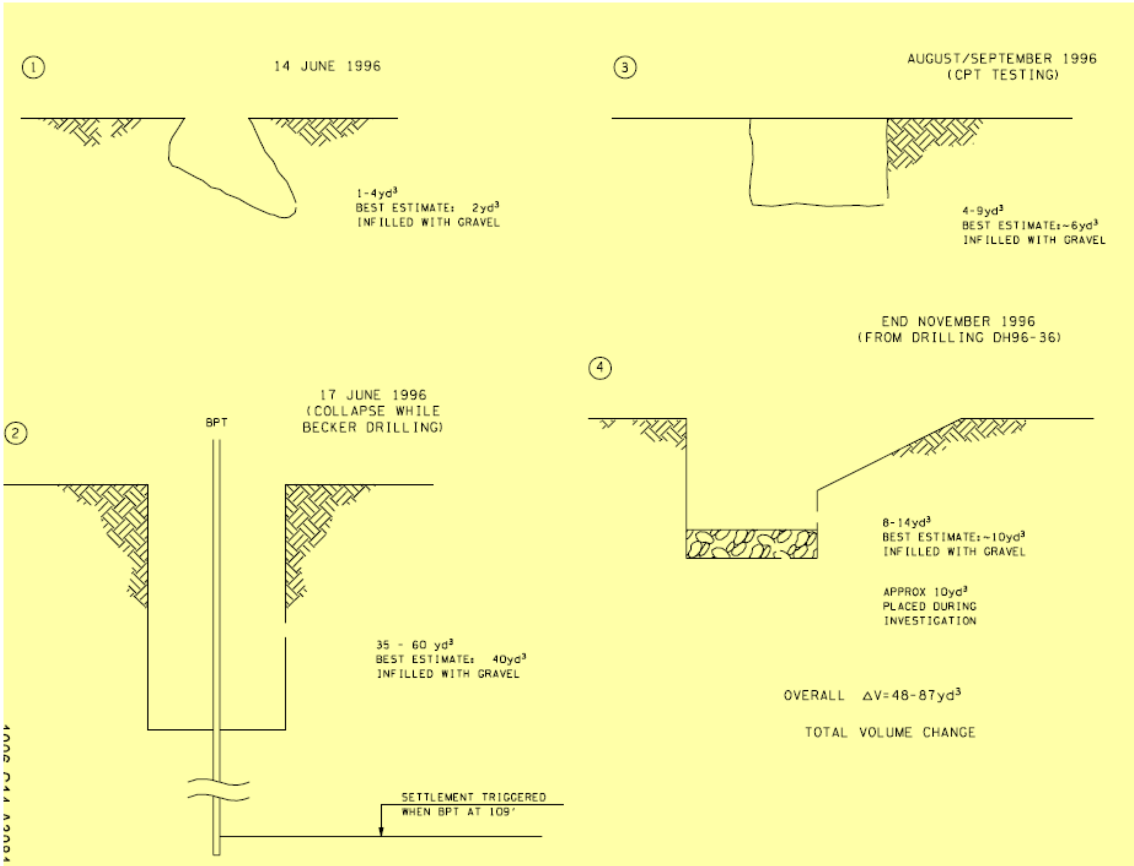


Figure H1. Evolution of Sinkhole No 1 (Figure 2-2, MEP 400)

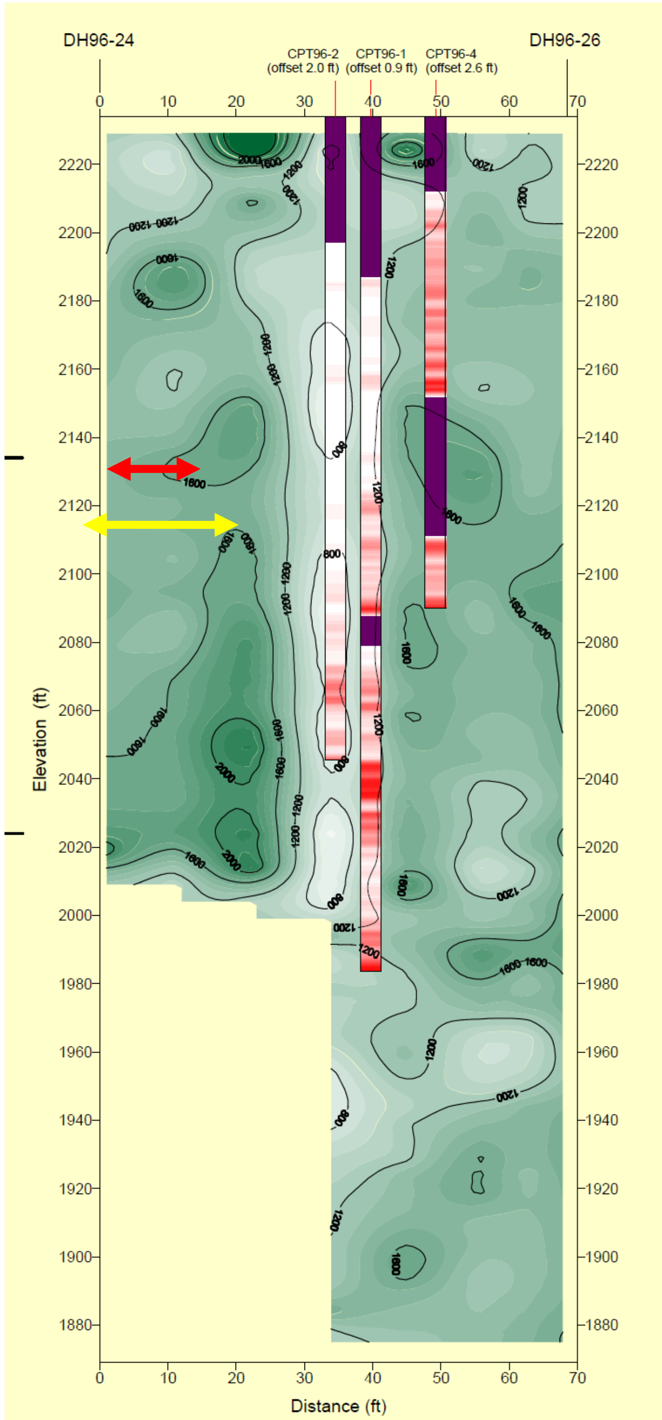


Figure H2. Sinkhole No 1 Seismic Tomography and Cone Penetration Test Data from Figure 2-6, Report MEP 400. Red arrowed dimension is approx. sinkhole diameter. The yellow arrowed dimension is approx. area of lower velocity.

The sinkhole was filled but further settlements totalling 6 to 8 ft occurred during CPT and drilling. Hence the total settlement was 33 to 36ft, and volume 48 to 87 cu.yd, best estimate 58 cu.yd.

H3.2 Sinkhole No 2 at Benchmark 2

The geometry of sinkhole No 2 is described in Section 3.1.2 of MEP 400. The sinkhole formed during drilling of a sonic drill hole on the dam crest nearby. Survey showed a 20 ft wide zone of settlement extending from the crest of the dam to 10 ft upstream of the benchmark. Total vertical maximum movement was 4 feet. There was also some horizontal movement which was attributed to shallow instability.

There are no equivalent figures to Figure H2 in the report.

H4 DEGREE OF COMPACTION AROUND BENCHMARKS

H4.1 Specified Degree of Compaction and Compaction Method

The specification for Zone 1 Core required that the maximum placed layer thickness was 10 inches (250mm). It was compacted by 100ton pneumatic rollers. The minimum acceptable compaction was 95% SMDD and the minimum desirable average was 98% SMDD (Low and Lyell, 1967).

Section 2.4 of Report MEP 400 indicates that deflector berms were constructed around the Benchmarks to protect them from construction equipment. Figure H3 shows these for Benchmark 1 when the Core was at about EL 1895ft.

Section 2.4 of MEP 4000 indicates that “construction records indicate that the backfill in this diamond shaped area with estimated dimensions of 45ft by 30 ft was compacted by walk-behind equipment”. It is apparent from the tyre marks that the 100 ton roller used for the Core was not used within this area.

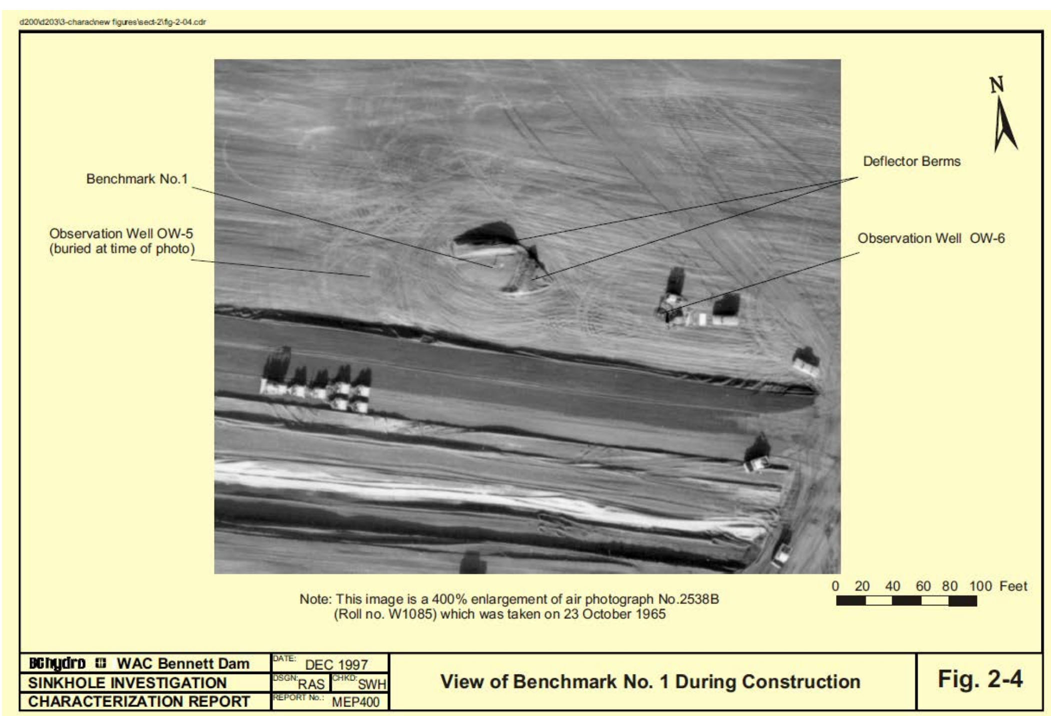


Figure H3. View of Benchmark No 1 during construction showing the “deflector berms”.

Figure H4 shows construction around Benchmark 2. The scale of the photograph makes it difficult to determine what measures were taken to protect the benchmark or compact around it. However the special treatment around the Observation Wells can be seen.

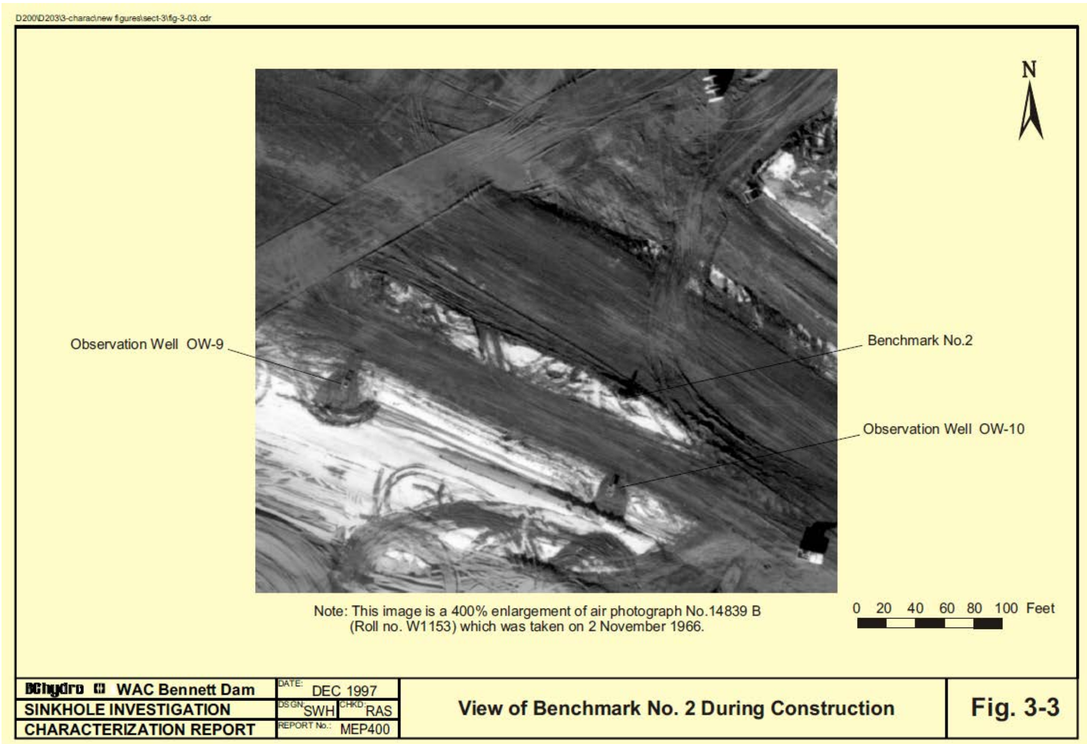


Figure H4. View of Benchmark No 1 during construction.

H4.2 Degree of Compaction Assumed for Report MEP 400 Analyses of Collapse Compression, and construction records of compaction

The degree of compaction of the Zone 1 Core surrounding the benchmarks is discussed in Section E5.3, Appendix E of MEP400A.

The probability distribution of % standard maximum dry density assumed for the analyses of collapse settlement is as shown in Figure H5.

This was based on:

- The “one compaction test from the zone around the benchmark during construction”, which was 93.8%.
- Dr R. Peck advising that the shortfall of 6 pcf of the average compaction around benchmarks was reasonable based on his experience. This gave 94%.
- A “minimum credible compaction” of 88%.

The database of compaction results for the Core near the two benchmarks was searched by BC Hydro and produced the results shown in Table H1. This shows that:

(a) There were very few tests carried out to check the degree of compaction of the backfill. For Benchmark 1 there were only 5 tests close to the benchmark in 265 ten inch lifts. For Benchmark 2 there were none. There were only 3 other tests for each benchmark within the larger area.

(b) For Benchmark 1 the lowest test result was 90.8%, not 93.8 % as assumed in the EP 400 report.

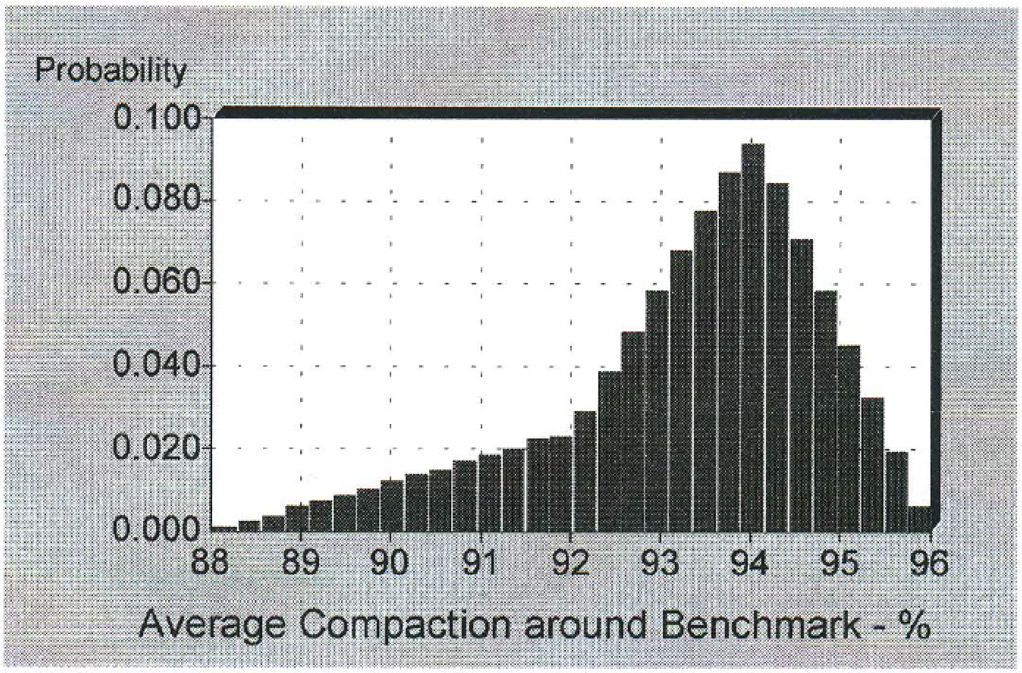


Figure H5. Probability distribution of average compaction around Benchmark 1as a % of standard maximum dry density.

Table H1. Results of compaction tests carried out during construction in the vicinity of Benchmarks 1 and 2

Benchmark	Degree of Compaction, % SMDD			
	Within 10ft STN and 8 ft U/S, D/S		Within 10ft to 20ft STN and 8 ft to 15 ft U/S, D/S	
	Elevation (ft)	% Compaction	Elevation (ft)	% Compaction
Benchmark 1	1886 1904 2008 2106 2107	97.7 93.4 96.8 90.8 101.4	1852 1919 1926	99.7 96.1 99.9
Benchmark 2	No tests		1994 2042 2120	99.3 96.0 96.0

H4.3 Degree of Compaction in Sinkhole No 1 and Surrounding Core from drill hole samples

Figure H6 shows the void ratio measured from samples in drill holes through the sinkhole at Benchmark No 1 and in the surrounding soil described as disturbed in report MEP 400.

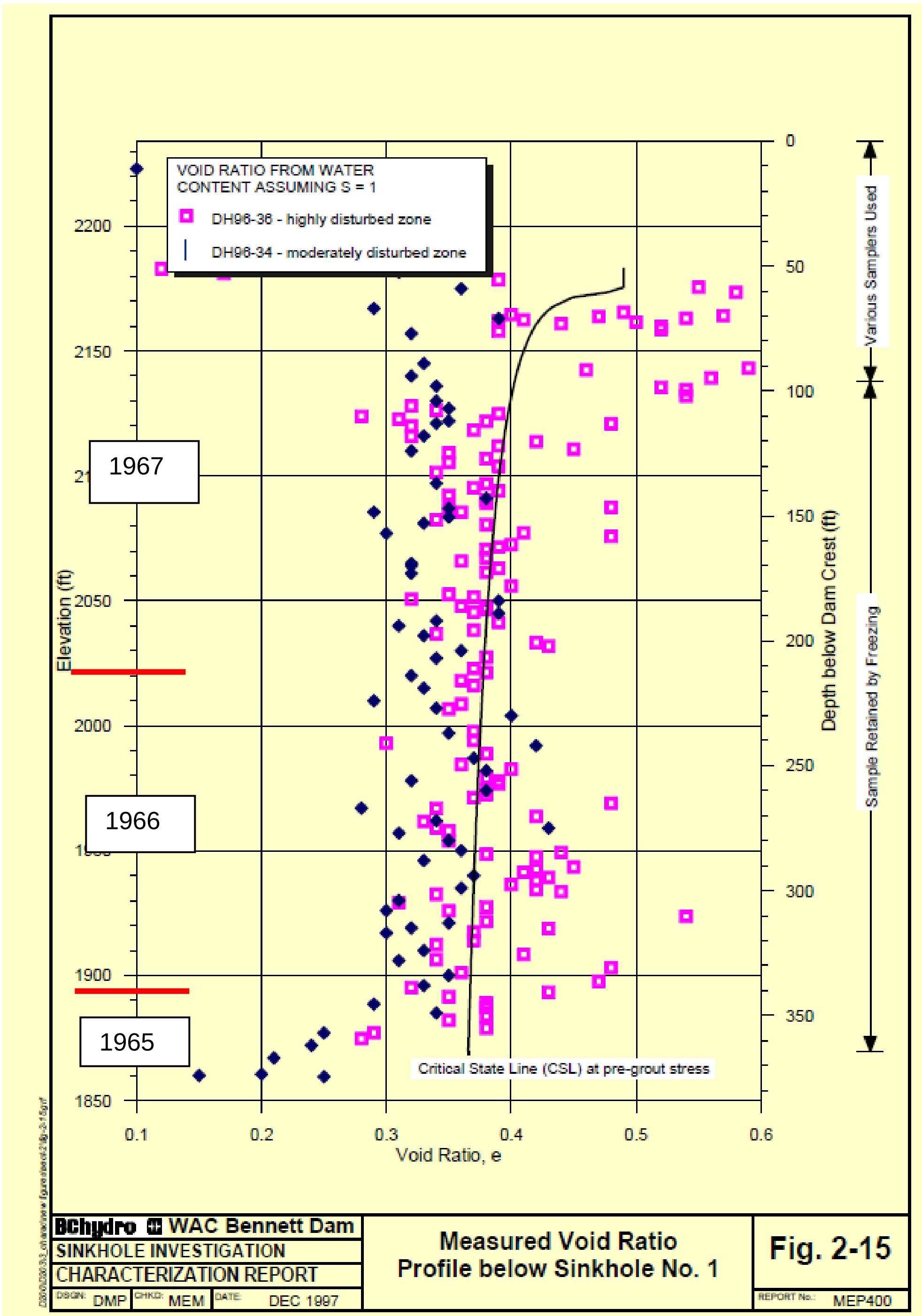


Figure H6. Measured void ratios below sinkhole No 1 at Benchmark No 1.

Table H2 summarizes this information and relates the void ratio data to equivalent dry density ratio and relative densities. To do this it has been assumed that:

Standard maximum dry density = 129.3 pcf, or 2.10 t/m^3 (MEP400, Section E5.3)

Soil Particle Density = 2.68 (MEP 400 Appendix D3)

Maximum void ratio = 0.678 (MEP 400, Section 2.7.6)

Minimum void ratio = 0.178 (MEP 400, Section 2.7.6)

Table H2. Measured Void Ratios and equivalent dry density ratio and relative densities

Description	Void Ratio range for most data	Equivalent Dry Density Ratio %	Equivalent Relative Density %
Sinkhole No 1, 50ft to 110ft	0.4 to 0.6 Median 0.51	91 to 80 Median 84	55 to 16 Medium Dense to Very Loose
Sinkhole No 1, 110ft to 260ft	0.3 to 0.4 Median 0.38	98 to 91 Median 92	76 to 55 Very Dense to Medium Dense
Sinkhole No 1, 260 ft to 340 ft	0.33 to 0.45 Median 0.4	96 to 87 Median 91	70 to 45 Dense to Medium Dense
Surrounding Sinkhole No 1 50ft to 350 ft.	0.3 to 0.39, Median 0.34	98 to 92 Median 95	76 to 58 Very Dense to Dense Median 67 Dense
96% SMDD	0.328	96	70 Dense
94% SMDD	0.351	94	64 Dense-Medium Dense
92% SMDD	0.387	92	58 Medium Dense - Dense
90% SMDD	0.418	90	52 Medium Dense
88% SMDD	0.44	88	48 Medium Dense

The following points are noted:

(a) The void ratios in the “moderately disturbed zone” or “Surrounding Sinkhole No 1, 50ft to 350 ft” are generally consistent throughout the depth profile albeit with some scatter.
The median equivalent dry density ratio is 95% SMDD.

There are some higher void ratio samples but nothing greater than 0.45, or 87% SMDD compaction.

Below 1880 ft the void ratios are lower, coinciding with 1965 construction.

(b) The high void ratios / low density ratios in the sinkhole from 50 ft to 110 ft may be related to this being above the phreatic surface so the soils are not saturated facilitating collapse. The piezometric levels in Boreholes DH 96-36; DH 96-8 and DH 96-37 have maximum piezometric heads at about 2130ft. Standpipe piezometers in DH96-1, DH96-7 and DH 96-8 have higher levels, close to reservoir level, and fluctuating with reservoir level, but these may be a perched water table or reflect higher permeability strata in the upper part of the dam.
The degree of compaction in this zone is from medium dense to very loose. The median dry density ratio is 84%.

(c) The void ratios in the sinkhole from 110ft to 260ft are reasonably consistent with a median of 0.38, or 92% SMDD. From 260 ft to 340 ft the void ratios are higher with a median of 0.4, or 91% SMDD, but with void ratios as high as 0.45, or 87% SMDD.

Note that there is not an equivalent plot to Figure H6 for Benchmark No 2.

H4.4 Cone Penetration Test Data

Cone Penetration Tests were carried out at the two Benchmarks in the locations shown in Figures H7 and H8. Figures H9 and H10 show the Cone Resistance plots for the two areas sorted into those within the sinkholes, and those in the surrounding areas.

From these it can be seen that:

Benchmark No 1 Area

(a) CPT 96-1 and CPT96-2 in the sinkhole show very low Qt values to a depth of 110 and 130ft respectively. This is indicative of very loose conditions.
(b) Below these levels the Qt values are variable with relatively low Qt layers between high Qt value layers. CPT 96-1 has a low Qt zone from 220 ft to 235ft approx.
(c) CPT 96-5 and CPT96-6 are within the 20 ft diameter zone downstream of the benchmark. They show quite high Qt down to 125ft and 110 ft respectively indicating the Core is relatively dense. However at these levels there is a layer in each CPT of zero Qt approx. This coincides with where the level of the maximum phreatic surface was, as discussed in H4.3 (b), and may represent a very loose zone or even voids at the top of the collapsed Core material.
(d) Below this in CPT 96-5 there is a relatively high Qt zone to 200ft depth, which has lower Qt layers at about 10ft intervals. It is notable that mud loss was recorded at three of these low Qt layers indicating they are susceptible to hydraulic fracture. From 200ft to 225 ft approx. there are low Qt strata.
(e) In CPT96-6 which is only about 10ft from CPT 96-5, the Qt profile is quite different. There are low Qt values from 150ft to 240ft. Much of this zone has Qt values of about 1 to 1.5MPa, indicating very loose or loose conditions. There are strata at 170ft and 190ft with zero resistance. These conditions are looser than in CPT 96-1 and CPT 96-2 within the sinkhole.
(f) CPT 96-3 and CPT 96-4 which are about 13ft to 14ft from the benchmark show distinctly different conditions with very high Qt inter-layered with thin zones of 4MPa Qt at regular intervals of about 5ft elevation. In CPT 96-3 there are three layers at 135ft, 160ft and 175 ft approx. with Qt approaching zero.
(g) CPT 96-7 and CPT 96-8 show different conditions again, with relatively uniform Qt in the range 4 to 6 MPa from 50ft to 180 ft increasing to 10 to 15 MPa to the base of the tests. CPT 96-7 is about 14ft upstream of the benchmark and CPT96-8 is about 90 ft from the benchmark but near Observation Well OW-5.

Benchmark No 2 Area

(a) The probes were all cased to 100ft through the Shell so there is no data in the top 100ft.
(b) CPT 96-9 and CPT 96-10 are in the “sinkhole”. They show Qt of about 1 MPa from 100ft to 125ft and generally 1 MPa to 3 MPa, or very loose conditions from that level to the base of the hole. In CPT 96-10 there is distinct layering at regular 5ft intervals and this may reflect that the method of placement and compaction around the benchmark was in 5 ft layers with little compaction.
(c) CPT 96-11 and CPT 96-12 which are 13ft and 17ft respectively from the benchmark show conditions from 100 ft to 150 ft in CPT 96-11 and 130 ft in CPT 96-11 which are similar to CPT 96-5, with dense strata inter-layered at about 10 ft intervals with Qt 3 MPa strata. This continues to the base of the probe in CPT 96-11 but in CPT 96-12 there is quite low Qt from 130 ft to 200ft approx. CPT 96-13 which is about 11ft upstream of the benchmark has somewhat denser conditions than CPT 98-12, and is similar to the CPT upstream of Benchmark No 1. It does have a 5 ft section centred on 150ft with Qt about 1.5 MPa.
(d) Note that the piezometers in this area show piezometric levels of about 2150 ft to 2170 ft in piezometers in CPT 96-10 and CPT 96-11, but much lower in CPT 96-12 and CPT 96-13

H4.5 Compaction Grouting - Grout Takes

Figures H11 and H12 show the grout takes for the two Benchmark areas.

It can be seen that:

Benchmark No 1

- The grout take from the surface to 100ft is lower than at greater depths. This is despite the sinkhole being lower density in the 50 to 100ft range. The grout takes were quite variable in each grout hole. The lower takes may be related to the fact lower pressures were used in the upper 150 ft than lower down.
- The lower grout takes below 260 ft do not seem to correlate with void ratios in the sinkhole or in strata surrounding the sinkhole.
- The high grout take at 140 ft may be related to the low Qt values in CPT probes, but that is not clear as there are other low Qt zones (Figure H11).

Benchmark No 2

- The grout takes are lower in the upper 75ft. This may be due to the significantly lower grout pressures used and to this part being in Shell, not Core material
- Grout takes below this are similar to Sinkhole No 1 despite the CPT data indicating sinkhole No 1 was lower density than sinkhole No 2 (Figure H12). From this it seems that the grout take is significantly controlled by the strata around the 10 ft diameter sinkhole rather than those in the sinkhole.

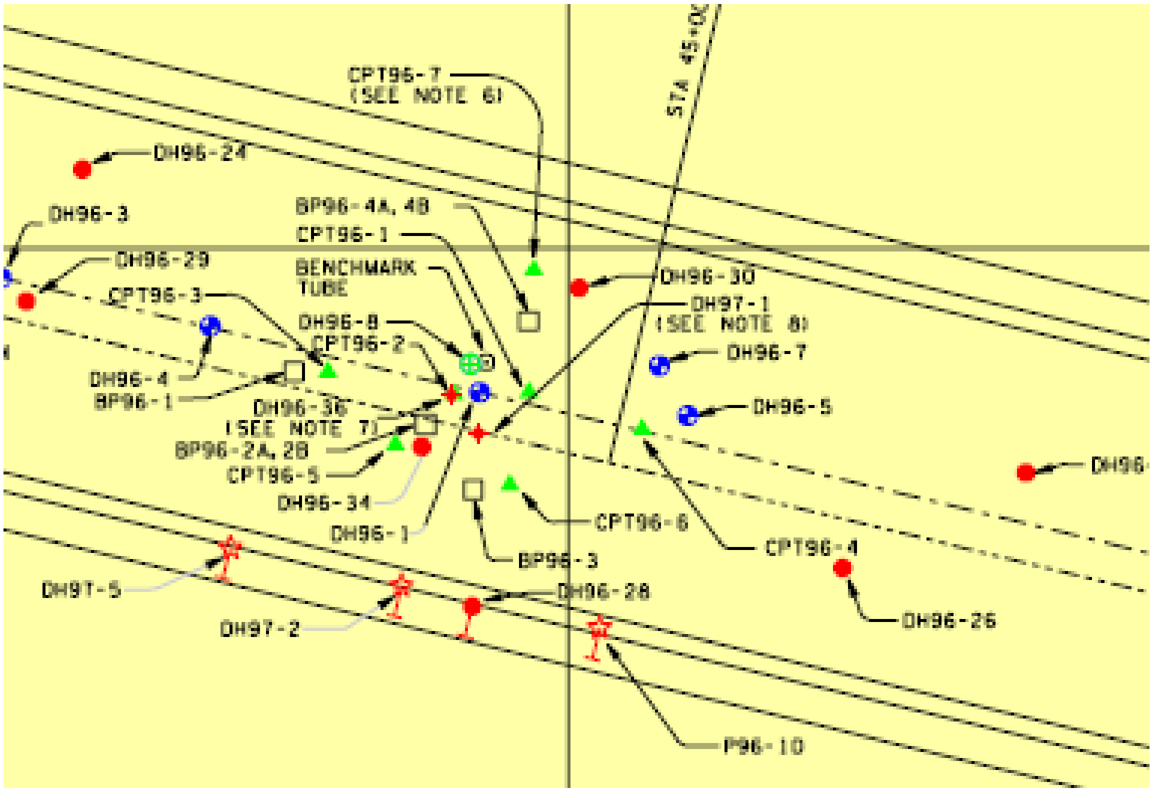


Figure H7. Location of Cone Penetration Tests and Boreholes around Benchmark No 1 sinkhole

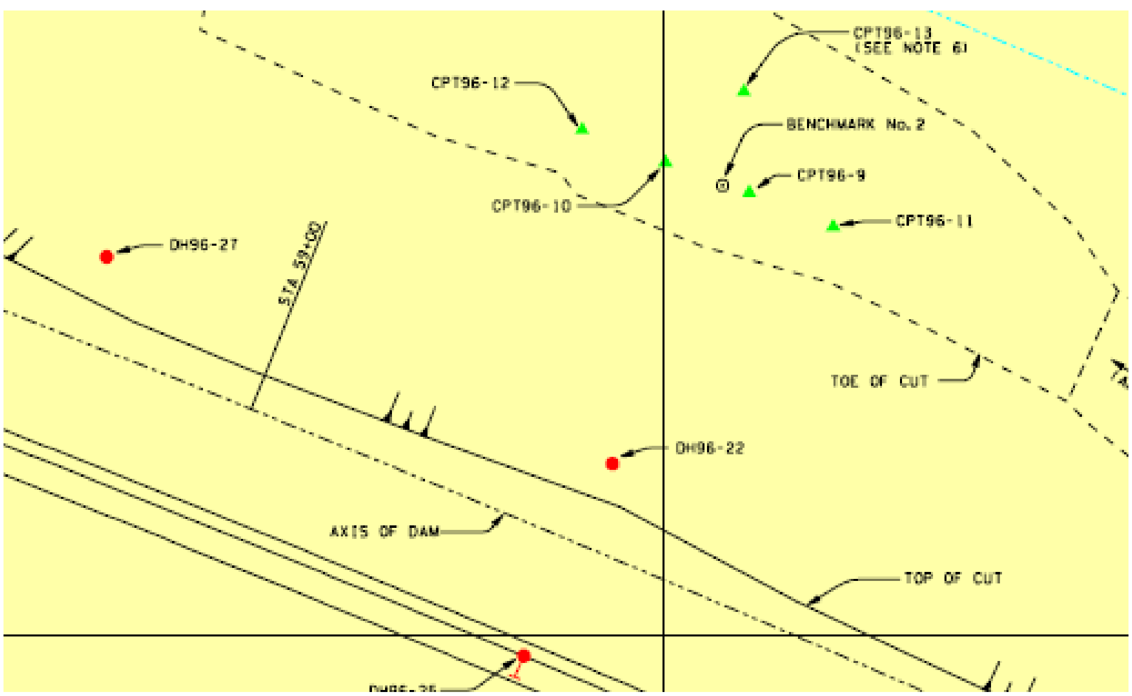
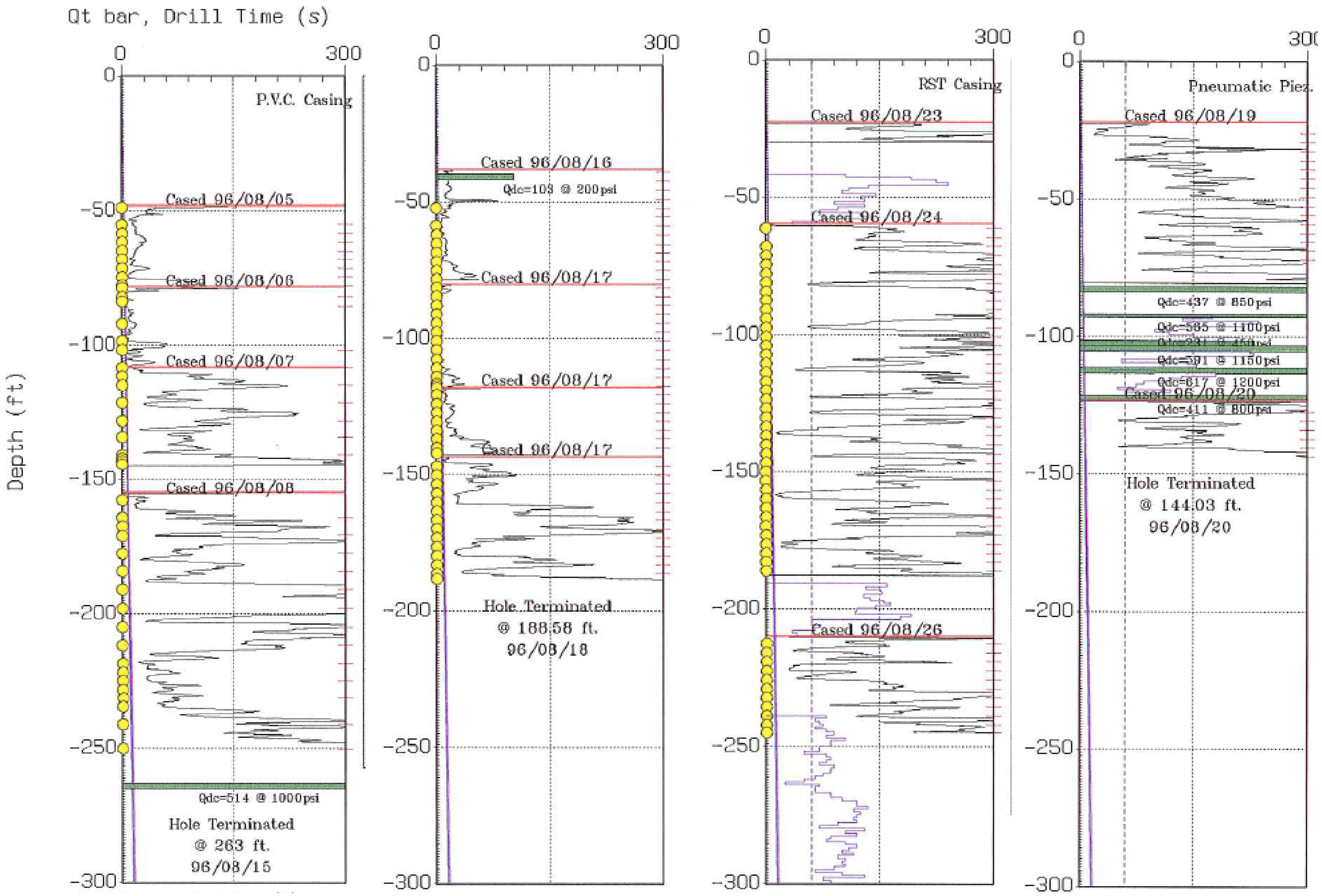


Figure H8. Location of Cone Penetration Tests and Boreholes around Benchmark No 2 sinkhole



CPT 96-1 CPT 96-2 CPT 96-3 CPT 96-4

Figure H9a. Cone resistance for CPTU tests at Benchmark No 1

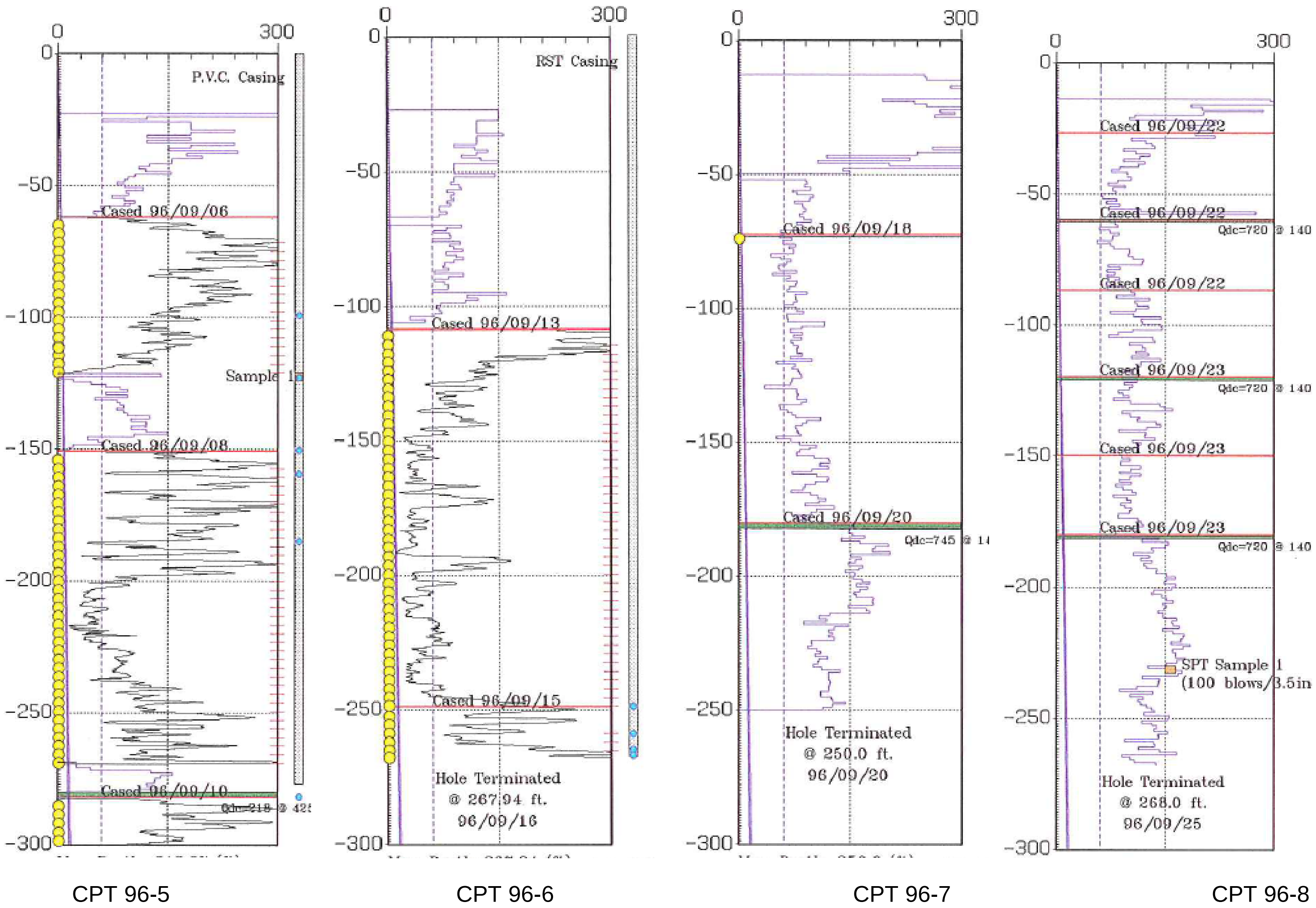


Figure H9b. Cone resistance for CPTU tests at Benchmark No 1

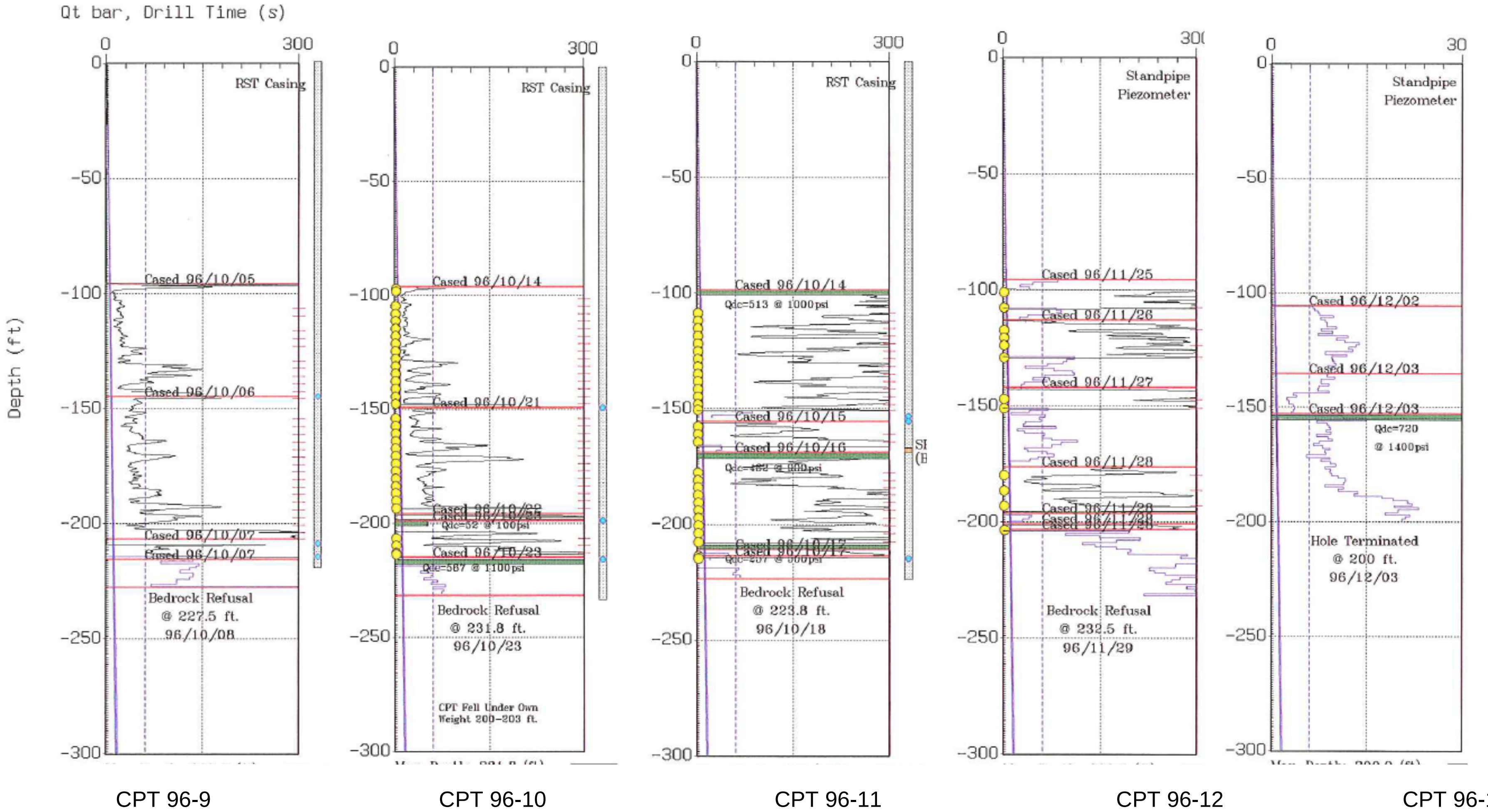


Figure H10. Cone resistance plots for CPTU tests at Benchmark No 2

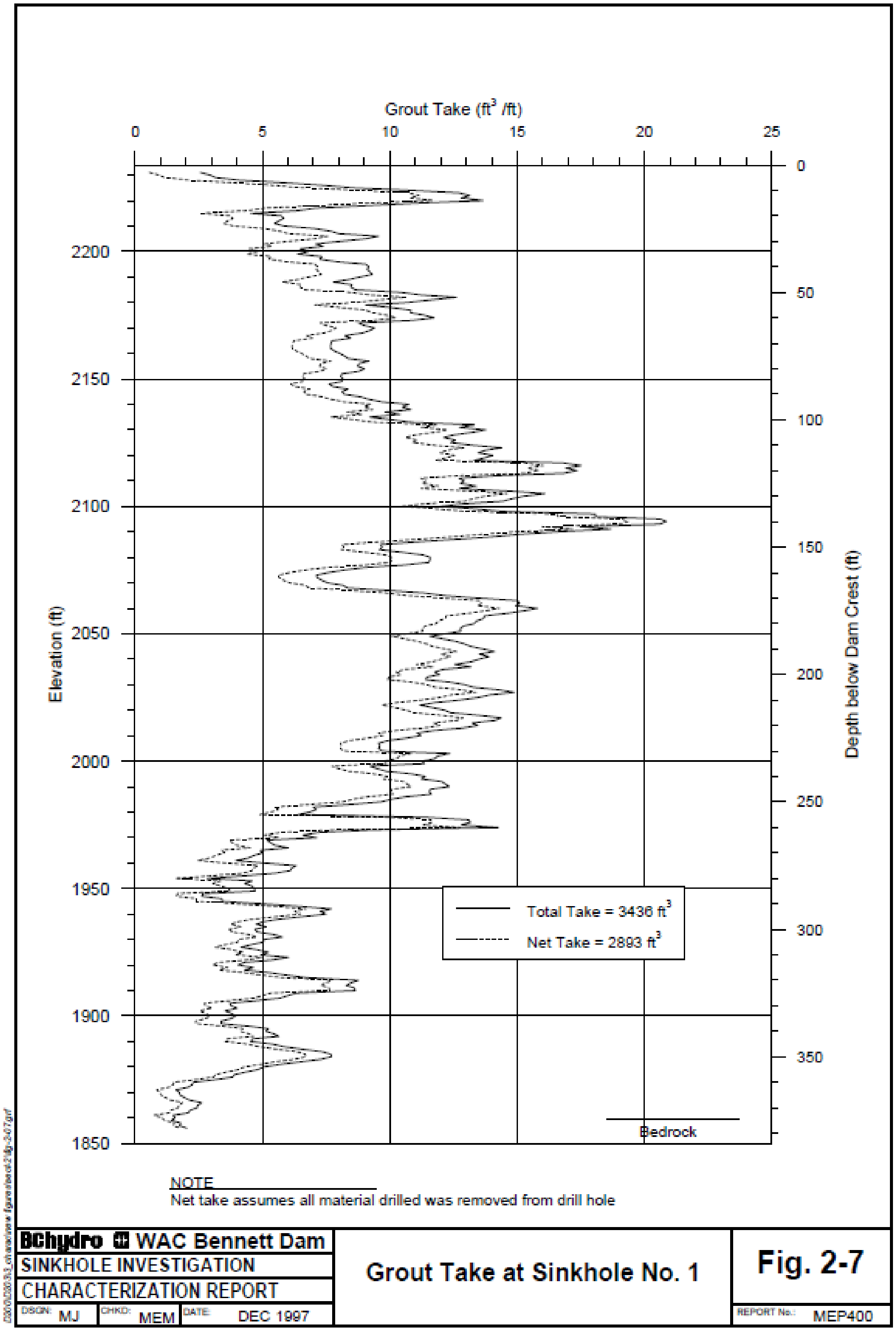


Figure H11. Compaction Grout Takes around Benchmark No 1.

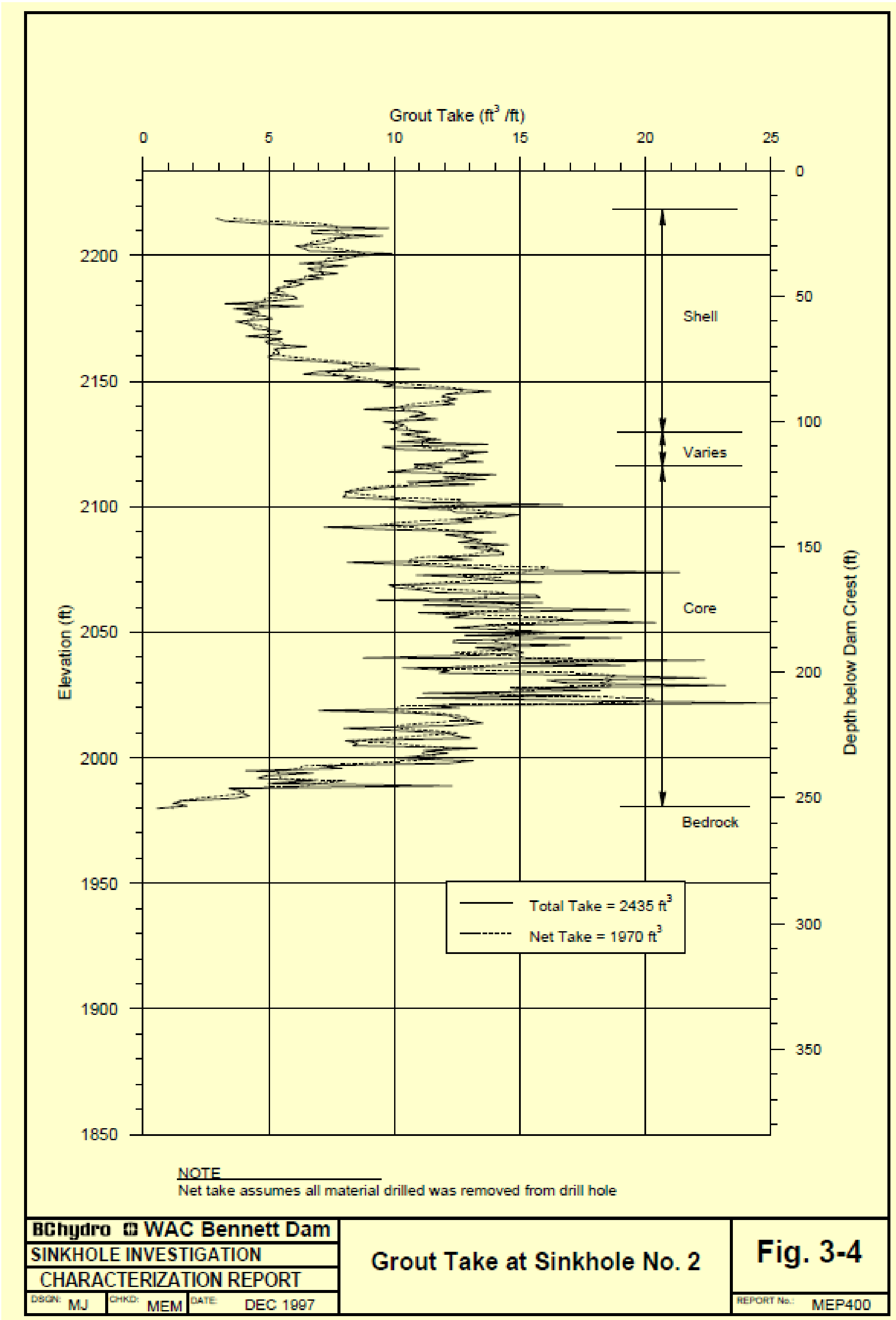


Figure H12. Compaction Grout Takes around Benchmark No 2.

H4.6 Wetting Settlement Laboratory Test Data

Figure H13 shows the results of wetting settlement laboratory tests. These are discussed in Section 2.7.5 of Report MEP 400.

From the tests, taking the relationship between void ratio and density ratio given in Table H2, it could be concluded that for materials compacted to moisture contents between 5% and 7%:

- A volumetric strain of between 0.1% and 0.5% would be expected for soils wetted at 25kPa consolidation pressure, and initial compaction between 88% and 98%.
- Volumetric strains of between 1% and 3% would be expected for samples wetted at a consolidation pressure of 1600 kPa and initial compaction between 92% and 94%.
- Volumetric strains of up to 5% are possible for samples wetted at 1600 kPa consolidation pressure and 84% compaction, at 5% moisture content.
- Volumetric strains of up to 8% are only possible for samples wetted at 1600 kPa consolidation pressure and 80% compaction, at 5% moisture content.

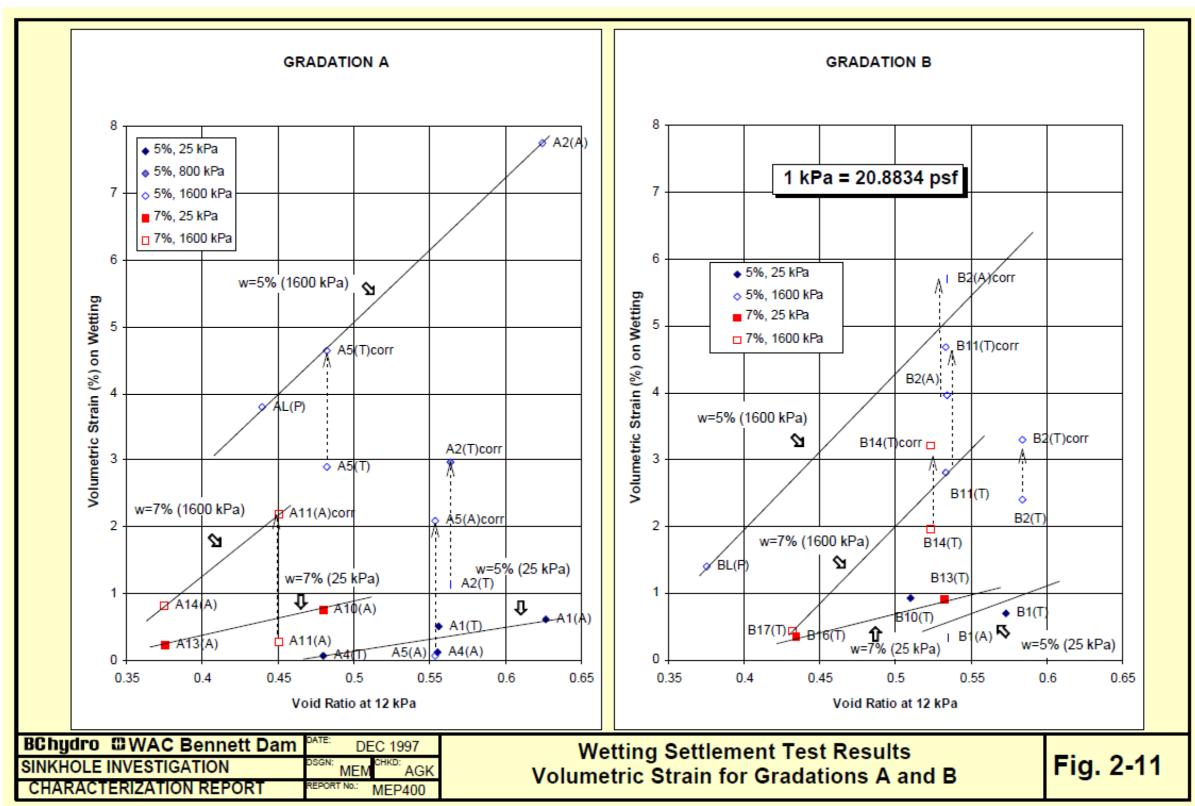


Figure H13. Volumetric strains on wetting from laboratory tests

H4.7 Foundation Geometry and Effect on Stresses in the Dam

Figures H14 shows the longitudinal profile of the foundation along the dam centreline at Sinkholes No 1 and No 2. Figures H15 and H16 show the foundation profiles in upstream-downstream cross-sections at Sinkhole No 1 and Sinkhole No 2 respectively.

The longitudinal profile at Sinkhole No 2 is such that extension strains are highly likely at the Sinkhole. At Sinkhole No 1 the cross section profile would also make extension strains highly likely.

These could have a significant part to play in whether concentrated leak erosion occurs through cracks or hydraulic fractures in low stress zones subject to hydraulic fracture.

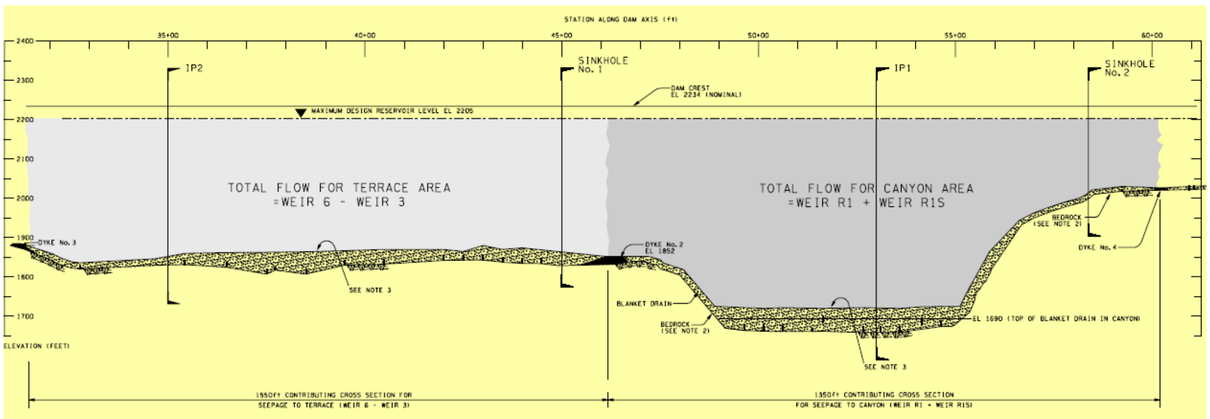


Figure H14. Longitudinal profile along dam axis

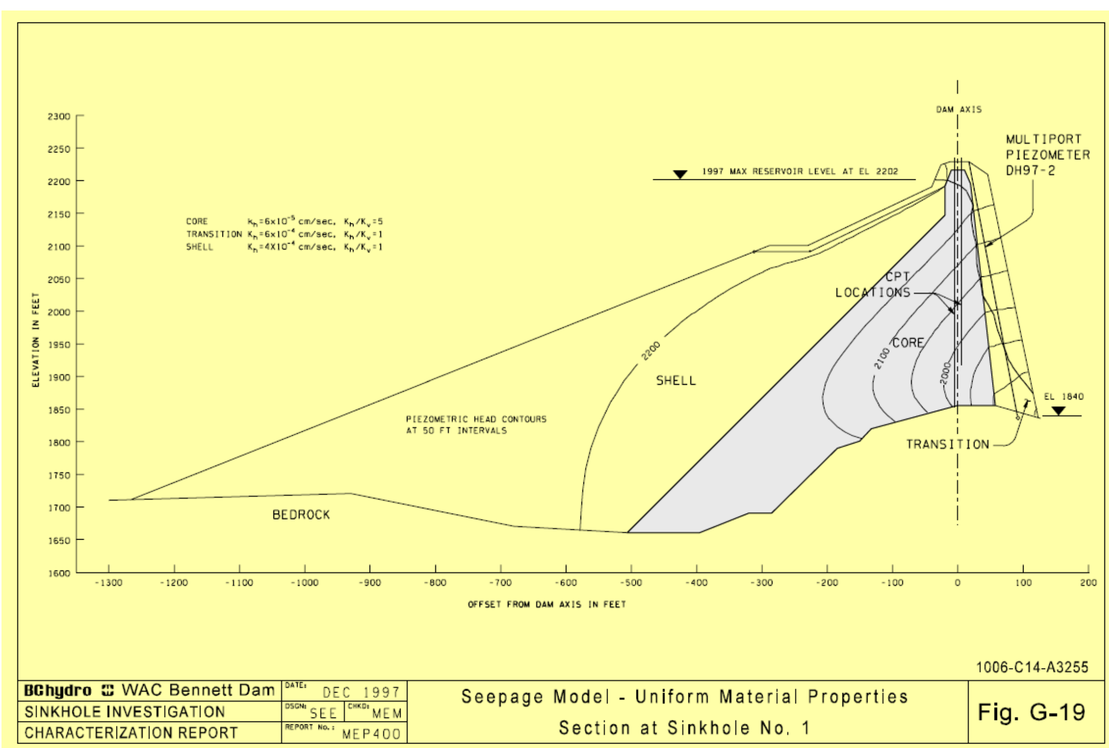


Figure H15. Cross section at Sinkhole No 1

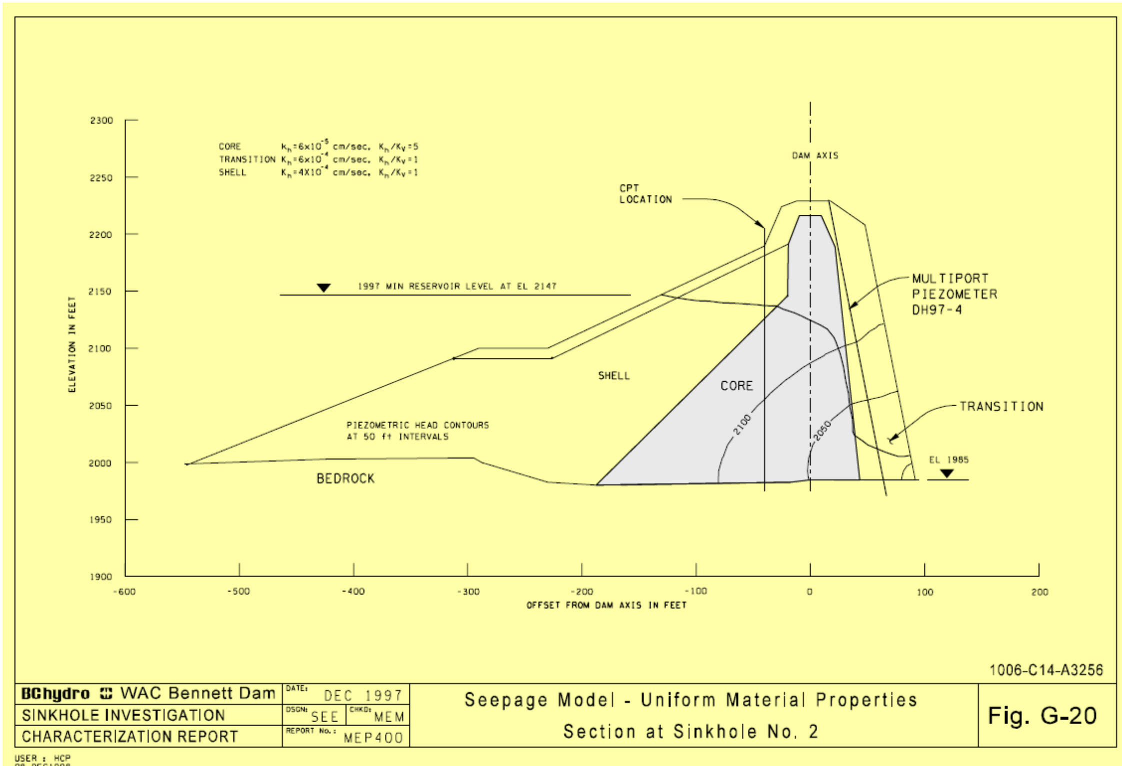


Figure H16. Cross section at Sinkhole No 2

H4.8 Gradation trends at Sinkhole No 1

In Section 2.7.2 of MEP 400 there is a discussion about the presence of low fines content samples taken from DH96-34 in the "moderately disturbed" zone around Sinkhole No 1. This refers to the data in Figure 2-10 of that report.

A review of the log of that borehole which was drilled by sonic drilling shows that:

- The zone of low fines content from 145 ft to 151ft was in an area where there had been a number of attempts to clean out the hole, with material moving up to 7ft into the casing each attempt. The casing also dropped under its self weight from 145ft to 150ft, and was then pulled back to 145 ft apparently prior to drilling and sampling from 145 ft to 150 ft. Given this, it is questionable whether the gradations are representative of the material in-situ.
- The second low fines content zone was at about 280 ft. The run from 275ft to 285 ft had no recovery on the first attempt. It is probable that the sample fell through some depth of water in the hole resulting in the fines being washed from the sand as it fell through the water.

It is concluded that it is questionable at least whether low fines content zones exist as assumed in MEP 400.

H4.7 Discussion on the Likely Degree of Compaction Achieved around the Benchmarks

Benchmark No 1.

(a) It seems certain that the average degree of compaction around the Benchmark was lower than in the Core generally. It also seems certain that the degree of compaction was lowest in the 8ft to 10 ft diameter surrounding the benchmarks because this is where the sinkholes formed.

(b) From the shear wave velocities the ground for a diameter of about 20 ft is not as well compacted as the surrounding ground. This would be consistent with it being more likely that the ground further from the benchmark was covered by a roller, and for what the scant compaction data is worth, is consistent with that data.

(c) This area has been designated as moderately disturbed zone in Report MEP 400, but it might just as well be that there is no disturbance, and the zone was simply not as well compacted. The void ratios are consistent with depth which supports the argument that the properties are as placed rather than affected by the sinkhole or internal erosion. The median compaction is 95% SMDD in this zone.

(d) The CPT data presents a more complex picture for this area. It shows variable conditions in the “moderately disturbed zone” with much lower Qt values in CPT 96-6 than in CPT 96-5 and CPT 96-7 which are all within the 20 ft diameter area. This could be a result of poor compaction extending to CPT 96-6, or internal erosion resulting in effective loosening of the strata.

(e) The inter-layering of dense high Qt material with much lower Qt strata at regular elevation intervals seems to indicate this is a result of placement compaction variations, possibly emphasised by wetting compaction of the looser strata as it saturates. The presence of three near zero Qt layer in CPT 96-3 is consistent with wetting compaction of the saturated soil below this level leaving a very loose layer or void with a roof of dense soil above it. The layering is at about 5 ft intervals, or a week’s placement. It may be that there was a shift or day when there was no work and the surface desiccated by sunshine or freezing or that the deflector berms were raised in 5 ft lifts.

(f) The high void ratios / low degree of compaction from 50ft to 110ft in the sinkhole is in Core material which settled during drilling of the Becker drill hole in the sinkhole to give the 21 ft deep sinkhole. Hence the void ratios and degree of compaction would have been even lower before the drilling. The measured median void ratio was 0.51, equivalent to 84% SMDD. This zone is above the phreatic surface as shown by piezometers adjacent to the sinkhole and the lack of saturation helps explain the high void ratio / low density. The CPT also shows that the soils in the sinkhole are very loose down to 110ft / 130 ft.

(g) The presence of zero CPT Qt resistance at the 110ft / 130 ft depth is consistent with collapse settlement of the saturated soil below this level leaving a very loose layer or void with a roof of partially saturated soil above it.

(h) The median void ratio for the remainder of the sinkhole is about 0.39, or 91% to 92% SMDD. This is reasonably consistent with CPT 96-6.
The void ratio of this zone before saturation of the Core and collapse settlement was almost certainly higher. If it was at the void ratio now measured in the 50ft to 110ft section, i.e. 0.51 or 84% SMDD, then collapse settlement has resulted in 7% to 8% increase in density. If this is applied from 110ft to 360ft, it would result in 17 to 20 ft settlement. This is close to the observed depth of the sinkhole. However this does not account for any increase or decrease in volume from 50ft to 110ft, or any effects of movement of soil from the surrounding area into the sinkhole area.

(i) The question arises whether the Core around the benchmark could have been placed at an average density of 84% SMDD, or 33% relative density, i.e. loose. This would require no compaction being carried out. This is at odds with the statement in Section 2.4 of Report MEP 400 that the ground near the sinkhole was compacted by “walk behind equipment”, i.e. presumably a small roller in the same layer thickness as the Core (250mm). However unlike the backfilling around the instrument risers and the observation wells, BC Hydro has not presented the EEP with definitive information on the placement method around the benchmarks. The benchmarks were installed at the Contractors request, and there may have been less care taken to compact the Core around them. The lack of compaction control tests supports this view. The Memo prepared by BC Hydro dated 2nd February 2012 does not present any additional data.

(j) If the material placed around the benchmarks was compacted by a small roller it is unlikely that the bottom part of the layers were compacted much more than as placed. For such placement an average degree of compaction of 0.5 (96 + 88) = 92 % would be reasonable. This is based on such rolling achieving a maximum of 96% compaction at the surface, and no densification from the placed density of say 88% at the base.

(k) From these it is concluded that wetting compaction cannot alone explain the amount of settlement at Sinkhole 1 and that internal erosion must have contributed.

(l) The higher void ratios / lower density in the sinkhole from 260ft to 340 ft, the low Qt values from 150 ft to 250 ft in CPT 96-6, and in CPT 96-1 from 220 ft to 235 ft are difficult to explain by collapse compression. The high void ratios coincide with a widening of the 1200 ft/sec shear wave velocity in Figure H2 but there is no change in the shear wave velocity in the upstream –downstream direction in the Section through DH 96-30 and DH 96-28. The lower velocities in that section are probably due to stress relief effects of the embankment as a whole.

Benchmark No 2.

There is far less data relating to this area than for Sinkhole No 1. It appears that:

(a) There is a low density / high void ratio zone in the 10 ft approx diameter centring on the benchmark, and quite low density / high void ratio at some depths up to 17ft from the benchmark.

(b) There is distinct layering in loose to very loose strata at regular 5ft intervals in CPT 96-10 and this may reflect the method of placement and compaction around the benchmark was in 5 ft layers with little compaction.

The grout takes in Sinkhole No 2 was similar to Sinkhole No 1 despite the much smaller sinkhole depth. This may indicate that the grout take was related more to compacting the ground around the sinkholes rather than in the sinkholes themselves, and that the compaction was deficient around both benchmarks.