FORTIS BC™

Diane Roy
Director, Regulatory Services

Gas Regulatory Affairs Correspondence
Email: gas.regulatory.affairs@fortisbc.com

Electric Regulatory Affairs Correspondence
Email: electricity.regulatory.affairs@fortisbc.com

FortisBC
16705 Fraser Highway
Surrey, B.C. V4N 0E8
Tel: (604) 576-7349
Cell: (604) 908-2790
Fax: (604) 576-7074
Email: diane.roy@fortisbc.com
www.fortisbc.com

March 12, 2015

Via Email
Original via Mail

British Columbia Public Interest Advocacy Centre
Suite 209 – 1090 West Pender Street
Vancouver, B.C. V6E 2N7

Attention: Ms. Tannis Braithwaite, Executive Director

Dear Ms. Braithwaite:

Re: FortisBC Energy Inc. (FEI)

Application for a Certificate of Public Convenience and Necessity (CPCN) for Approval of the Lower Mainland Intermediate Pressure (IP) System Upgrade (LMIPSU) Projects (the Application)

Response to the British Columbia Public Interest Advocacy Centre representing the British Columbia Old Age Pensioners' Organization, Active Support Against Poverty, Disability Alliance BC, Council of Senior Citizens' Organizations of BC, and the Tenant Resource and Advisory Centre *et al.* (BCOAPO) Information Request (IR) No. 1

On December 19, 2014, FEI filed the Application referenced above. In accordance with the British Columbia Utilities Commission Order G-1-15 setting out the Regulatory Timetable for the review of the Application, FEI respectfully submits the attached response to BCOAPO IR No. 1.

If further information is required, please contact the undersigned.

Sincerely,

FORTISBC ENERGY INC.

Original signed:

Diane Roy

Attachments

cc: Commission Secretary
Registered Parties (e-mail only)

1. APPLICATION

Reference: Exhibit B-1 /Section 1 / pg.5

1.1. Please provide the relevant sections of the *British Columbia Oil and Gas Activities Act* [in addition to the noted sections 37(1) and 37(3)] and the Canadian Standards Association Standard CSA-Z662-11 which FEI believes it may be in contravention of if the proposed projects are not undertaken.

Response:

There may be sections other than 37(1) and 37(3) of the British Columbia *Oil and Gas Activities Act* that FEI may be found to be in contravention of, if the applied for Projects are not undertaken. For instance, section 3(1) of the Pipeline Regulation under the British Columbia *Oil and Gas Activities Act* requires pipelines to be operated and maintained in accordance with CSA Z662. Another example is that if the Fraser Gate IP Project is not undertaken, the Company may be found in contravention of the following clause from the section 6(1) of the Pipeline Regulation under the British Columbia *Oil and Gas Activities Act*, which states that, "*If a pipeline is being or has been constructed across, along, over or under a public place or the right of way of a highway, road, railway, underground communication or power line or other pipeline, the pipeline permit holder must: (a) take all reasonable steps so as not to endanger public safety or the environment.*"

The relevant clause from CSA Z662-11 that FEI believes that it may be found in contravention of if the Coquitlam Gate IP Project does not proceed is Clause 12.10.2.3 (d), which states that, *"Where the condition of distribution or service lines, as indicated by leak records or visual observation, deteriorates to the point where they should not be retained in service, they shall be replaced, reconditioned, or abandoned."*

The relevant clauses from CSA Z662-11 that the Company may be found in contravention of if the Fraser Gate IP Project does not proceed are:

- Clause 10.3.1.1, which states that, *"Where the operating company becomes aware of conditions that can lead to failures in its pipeline systems, it shall conduct an engineering assessment to determine which portions can be susceptible to failures and whether such portions are suitable for continued service."*
- Clause 10.3.1.3, which states that, *"Where the engineering assessment indicates that portions of the pipeline system are susceptible to failures, the operating company shall*

either implement measures preventing such failures or operate the system under conditions that are determined by an engineering assessment to be acceptable."

1.2. Please explain the implications of an OGC finding that FEI has failed to comply with these sections of the *British Columbia Oil and Gas Activities Act* and/or the Canadian Standards Association Standard CSA-Z662-11

Response:

Consequences for failing to comply with provisions of the British Columbia *Oil and Gas Activities Act* (the Act) and/or the Canadian Standards Association Standard CSA-Z662-11 are set forth in Part 5 of the Act, which addresses compliance and enforcement. As examples, Part 5 contains the following provisions:

> **49** (1) An official may, in writing, issue to a person carrying out an oil and gas activity or a related activity an order under this section with respect to those activities or any of the person's obligations under the Act or the regulations or the person's permit or authorization, if any, if, in the opinion of the official,
>
> (a) the person fails to comply with the Act, the regulations, a previous order made under the Act, or the person's permit or authorization, or
>
> (b) the order is necessary
>
> (i) to mitigate a risk to public safety,
>
> (ii) to protect the environment, or
>
> (iii) to promote the conservation of petroleum and natural gas resources.
>
> …
>
> (4) Without limiting subsection (3) (b), an order under subsection (1) may specify any of the following requirements:
>
> (a) that a person must apply to obtain or amend a permit or an authorization in accordance with the Act and the regulations;
>
> (b) that a person remedy a failure referred to in subsection (1) (a);
>
> (c) that a person repair damage to the environment;

> (d) that a person suspend or resume an oil and gas activity or any aspect of an oil and gas activity;
>
> (e) that a person use a specified method to carry out an oil and gas activity;
>
> (f) that a person conduct tests, take samples, conduct analyses and submit records and information to the commission;
>
> (g) that a person control or prevent the escape of petroleum, natural gas, water, waste or other substances from a well, pipeline or facility;
>
> **62** (1) After giving an opportunity to be heard to a person who is alleged to have contravened a provision of the Act, the regulations, a permit, an authorization or an order, the commission may find that the person has contravened the provision.
>
> (2) If a corporation contravenes a provision referred to in subsection (1), a director, agent or officer of the corporation who authorized, permitted or acquiesced in the contravention also contravenes the provision.
>
> (3) If an employee, contractor or agent of a permit holder contravenes a provision referred to in subsection (1) in the course of carrying out the employment, contract or agency, the permit holder also contravenes the provision.
>
> (4) If a person contravenes a provision referred to in subsection (1), any other person who
>
> (a) is directly or indirectly responsible for the act or omission that constitutes the contravention, and
>
> (b) is a contractor, employee or agent of the person or of an other person described in paragraph (a) also contravenes the provision.
>
> **63** (1) If the commission finds that a person has contravened a provision referred to in section 62 (1), the commission may impose an administrative penalty on the person in an amount that does not exceed the prescribed amount.

Additionally, the following provisions from Part 8 of the Act, which addresses offences and court orders, may be applicable as well.

> **86** (1) A person who contravenes section 21, 35 (1), 36 (1), 37 (1) or (2), 39 (3), 40, 61 or 81, or in relation to an order issued under section 49, section 82, commits an offence and is liable on conviction to a fine not exceeding $1 500 000 or to imprisonment for not more than 3 years, or to both.

> (2) A person who contravenes section 35 (3) commits an offence and is liable on conviction to a fine not exceeding $1 000 000 or to imprisonment for not more than 2 years, or to both.
>
> (3) A person who contravenes section 34, 38 (1) or 39 (1), or in relation to an order issued under section 53 (2) (a), section 82, commits an offence and is liable on conviction to a fine not exceeding $500 000 or to imprisonment for not more than one year, or to both.
>
> (4) A person who contravenes section 35 (2) or 76 (1), or in relation to an order issued under a section not referred to in subsections (1) to (3) of this section, section 82, commits an offence and is liable on conviction to a fine not exceeding $100 000.
>
> (5) A person who contravenes section 37 (3) or 60 (1) or (2) commits an offence and is liable on conviction to a fine not exceeding $25 000.
>
> **88** (1) If the commission considers that a person is not complying, or has not complied, with an order issued under this Act, the commission may apply to the Supreme Court for either or both of the following:
>
> > (a) an order directing the person to comply with the order or restraining the person from violating the order;
> >
> > (b) an order directing the directors and officers of the person to cause the person to comply with or to stop violating the order.
>
> (2) On application by the commission under this section, the Supreme Court may make an order it considers appropriate.

Reference: Exhibit B-1/Section 1/pg. 9

1.3. Please provide the detailed calculation showing the derivation of the 3.39% delivery margin and 1.3% burner tip increases related to these projects.

Response:

The following two tables show the derivation of the 3.39% estimated delivery margin increase and the 1.3% burner tip increase.

Estimated Delivery Margin Impact

Line No.		2019 $000's	Reference
1	Incremental Revenue Requirement - Coquitlam Gate IP Project	$ 22,958	Confidential Appendix E-1-1, Schedule 1, Line 9
2	Incremental Revenue Requirement - Fraser Gate IP Project	1,588	Confidential Appendix E-1-2, Schedule 1, Line 9
3	Total	$ 24,546	
4	Amalgamated Delivery Margin	$ 723,603	1)
5	Estimated Margin Rate Increase	3.39%	Line 3 / Line 4

1) 2014 2015 Common delivery Rates and Delivery Rate Riders application, filed October 31, 2014, Appendix A Amalgamated Financial Schedules, Schedule 5, Line 28, Column 6 + Column 8

Estimated Burner Tip Impact

2019 Average Rate Impact / GJ	Annual Consumption GJ	Annual Bill Impact	Jan. 1, 2015 Burner Tip 3)	Impact on Burner Tip	
0.130	95	12.35	921.66	1.3%	2)

2) $12.35 / $921.66 = 1.3%

3) Original Interim Tariff Pages Effective January 1, 2015 to Reflect Amalgamation; BCUC Orders G-21-14, G-175-14, G-176-14, G-177-14, G-178-14; Tab 5, Page 1, Line 19, Column "Effective January 1, 2015 Rates – Annual $, filed December 8, 2014.

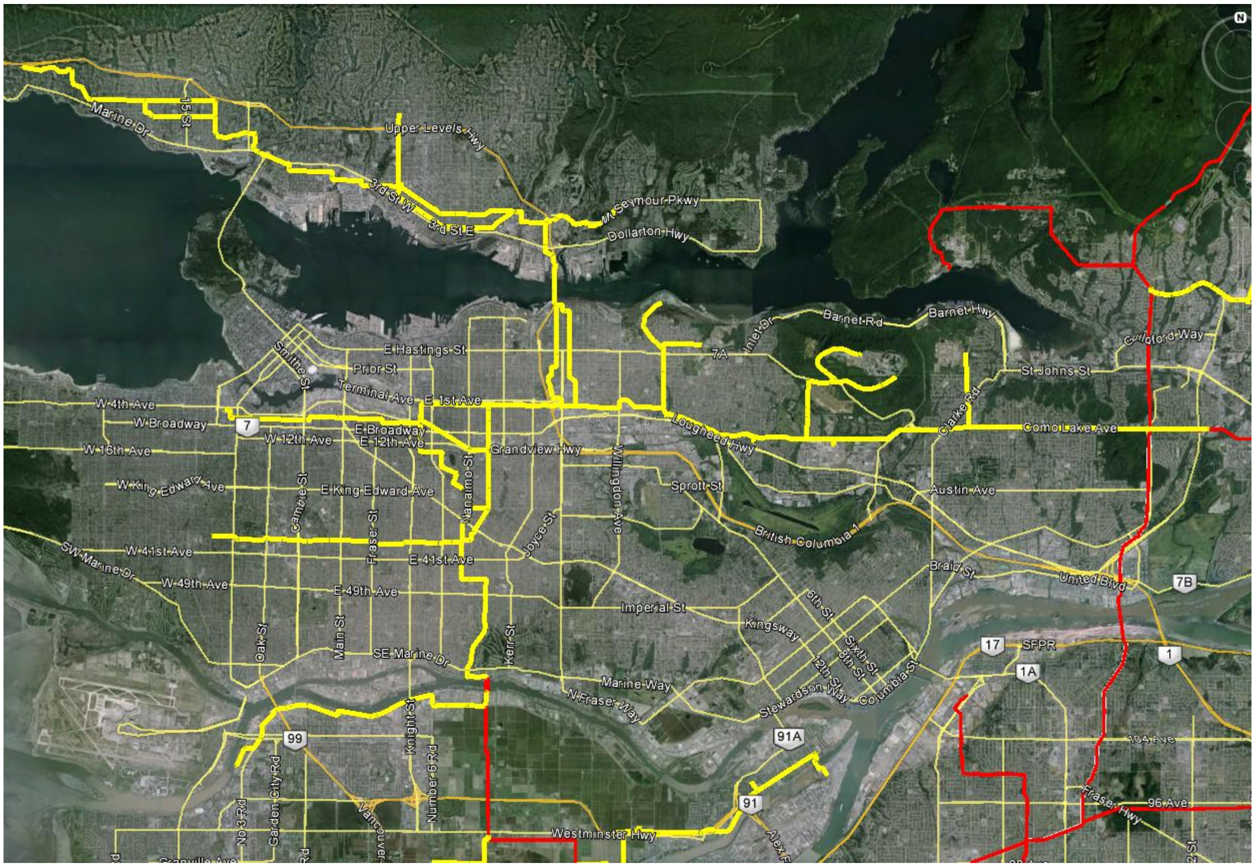
Reference: Exhibit B-1/Section 1/pg.4

1.4. The purpose of this interrogatory is to attain a better understanding of the routing and customers served along the routing of the Coquitlam Gate-Fraser Gate corridor. Please provide a map similar to that of Figure 1.2 but which shows:

- The major roads/arteries along the route of the existing and proposed new pipeline
- To allow greater detail shows only the areas bordered by the Fraser Gate Station, 2nd & Woodland Station, Coquitlam Gate Station, and Nichol Valve Station.
- Provides the names of the major pipeline lateral points (district stations) as shown in Figure 1.2 between the Coquitlam Gate Station-2nd & Woodland – Fraser Gate Station
- Divide the map into quadrants with quadrants defined by a major lateral or district station (for example using the segments identified Exhibit B-1/Appendix A-10/pg.6).
- For each quadrant showing the number of customers served off that respective pipeline segment.
- For each quadrant showing the peak delivered volumes (last full year period or 5 year average if available).
- Showing the number of leaks identified in each quadrant (as identified in response to Commission Staff interrogatory 1.1).

Response:

Please refer to Attachment 1.4 for a map showing the existing IP routing and major roads and arteries along the route and leak numbers along the pipeline segments. Please refer to the response to BCUC IR 1.6.1.1 for a map and table showing district station names, peak hour delivered volumes and approximate customers numbers for each delivery point.



Reference: Exhibit B-1/Section 1.3/pg.10

1.5. Indicate the frequency and duration of service disruptions caused by unplanned maintenance and repair of the Fraser Gate IP pipeline for the years 2010 – 2014.

Response:

There have been no unplanned maintenance and repair incidents on the Fraser Gate IP pipeline for the years 2010 – 2014.

1.6. Indicate the frequency and duration of service disruptions caused by unplanned maintenance and repair of the Coquitlam Gate IP pipeline for the years 2010 – 2014.

Response:

Please refer to the response to BCUC IR 1.1.1.2.

1.7. How have these service disruptions been dealt with in the past?

Response:

FEI's response to BCUC IR 1.1.1.2 contains the available recorded history of outages for the NPS 20 Coquitlam Gate IP pipeline. Please refer also to the response to BCUC IR 1.3.4 for a description of how these events have been managed operationally.

2. APPLICANT

No Questions

3. COQUITLAM GATE IP

Project Justification

Reference: Exhibit B-1/Section 3/

3.1. Please provide the monthly/annual leak reports for the Coquitlam IP. Please also provide any reports/presentations on these pipelines leaks for the period 2010-14 as provided to senior management of FEI.

Response:

Please refer to Attachment 3.1 for the requested 1) annual leak reports, and 2) reports and presentations to executive management.

1. Monthly leak reports provided to the BC Oil and Gas Commission, January 2014 through December 2014. No further reports have been filed, to date, in 2015. Additionally, a presentation to the OGC, dated October 15, 2013.
2. The presentations/reports on pipeline leaks to executive management during the requested time frame:
   - March 27, 2013: Presentation on Asset Management Strategy Document.
   - July 5, 2013 – an update from a director on the NPS 20" Coquitlam Gate IP pipeline condition to executive management.
   - Excerpts from 2013-2014 Quarterly Corporate Reports, reviewed by executive management and provided to the Board.

Reference: Exhibit B-1/Section 3.1.2.3.1/pg. 20

3.2. The evidence describes how the Coquitlam IP pipeline allows for the operational flexibility to do planned work on the Fraser Gate Station. Please provide the annual number and duration of Fraser Gate Station supply disruptions/closures for maintenance/force majeure that have occurred over the past 5 years and that required the Coquitlam IP pipeline provide supply (backfeed) in the Fraser Gate-Woodland Station corridor.

Response:

In the past 5 years, there have been no Fraser Gate station supply disruptions or closures that would have required the Coquitlam IP pipeline to provide support.

Alternative Solutions

Reference: Exhibit B-1/Section 1/pg. 6 & Exhibit B-1/Section 3.1.5/pg.29

3.3. Please provide the detailed calculation showing the derivation of the economic impact to the general public of failure at the Fraser Gate IP ($320 million) and the Coquitlam Gate IP ($64 million). Please show all assumptions in this calculation including duration of disruption. Please provide the high, medium and low case scenarios.

Response:

HJ Ruitenbeek Resource Consulting Ltd. provides the following response:

The methodology, assumptions and results relating to these figures can be found in the "Economic Consequence Analysis" Appendix A-5 (Exhibit B-1-1-1). Tables 3.1 and 3.2 in that Appendix summarise the cost impacts from incidents in different parts of the system. The Fraser Gate IP shows the greatest impact with a "Segment 1" failure resulting in a loss of $320.412 million. Of that amount, $141.78 million is attributable to costs borne by customers, while the balance is incurred by the utility through various costs (direct expenditures, relight costs, and revenue loss). Incidents throughout the Fraser Gate IP System will result in lower costs depending on where the incident occurs. An incident near the Coquitlam Gate Station would generate a loss of $181.95 million, of which $106.27 million is a "Residual" (described in Appendix A-5, Exhibit B-1-1) generally representing customers further north and not in the Metro IP system. In addition, modeling conducted for this work illustrated that approximately $12 million of this $181.95 million could be attributable to non-Metro IP customers such that the net impact would be approximately $64 million to the Coquitlam Gate IP customers. Table B1 in

Appendix A-5 shows, for example, that an incident at the inlet of Coquitlam Gate Station would potentially disrupt 163,280 customers, of which 41,400 are Metro IP customers.

The specific sensitivity analyses conducted for the IP System are summarized in Appendix A-5 Table ES-2a and Table ES-2b. The text accompanying that summary also confirms that "*…[in] the event of a failure in the Nichol to Port Mann segment, this corresponds to a net reduction in total consequences of approximately $64 million associated with these 41,400 customers.*" Inspection of the two tables shows that a Nichol to Port Mann incident would generate "As-Is" consequences of $192.63 million and "Residual" consequences of $128.91 million, for a net impact of $63.72 million.

Durations for these impacts are available in Appendix A-5 Table 3.3 or (in greater detail) in Table B2; these show the relight costs and duration of outages. Information relating to the location of outages is shown in Appendix A-5 Table B1. These illustrate that maximum outage times for a Fraser Gate IP (Segment 1) failure are approximately 20 days. A disruption at Coquitlam Gate could last 16 days for Coquitlam Gate IP customers.

Sensitivity tests are not conducted as "high, medium and low" but are provided for different sensitivity scenarios to demonstrate how different assumptions can impact results. These are discussed for all parts of the system in Appendix A-5 Section 4 and are summarized in Table 4.1. For example, the results show that the highest value is from "Sensitivity IV" corresponding to assumptions relating to colder weather (with gas demand 20% higher than otherwise assumed) and longer shutdown and relight times (additional 2 weeks to reference case). For a Segment 1 IP incident, this would increase impacts by 70% to about $543 million.

Reference: Exhibit B-1/Section 3.1.5/pg.29

3.4. Please provide the detailed calculation and the ranges for the estimate of the $64 million economic impact of a loss of supply on the Coquitlam Gate IP pipeline. Please show all assumptions in this calculation including duration of disruption. Please provide the high, medium and low case scenarios.

Response:

Please refer to the response to BCOAPO IR 1.3.3.

Reference: Exhibit B-1/Section 3.2.2/pg.41

3.5. The potential economic loss of a failure at Fraser Gate Station is noted at $320 million or $1871 per customer. We are unable to locate the derivation of this estimate, including the assumptions with respect to the estimated number of days/hrs of disruption and the estimate of utility lost revenue. Please provide the detailed calculation showing the derivation of the economic impact to the general public of failure at the Fraser Gate IP ($320 million). Please show all assumptions in this calculation including duration of disruption. Please provide the high, medium and low case scenarios.

Response:

Utility Lost Revenue is disaggregated and discussed in detail starting on page 18 of Appendix A-5 (Exhibit B-1-1). Please also refer to the response to BCOAPO IR 1.3.3.

Reference: Exhibit B-1, Table 3-1: Coquitlam Gate IP Project Non-Financial Comparison, pg. 41

Preamble: Alternative 3 in Table 3-1 is not considered a feasible alternative in part because it does not meet the objective to "provide sufficient operational flexibility." The footnote (1) in the notes section of the chart explains that this alternative would require "a bypass any time maintenance or repair is required."

3.6. Please explain why this alternative would require a bypass any time maintenance or repair is required.

Response:

Alternative 3 would replace the Coquitlam Gate IP pipeline with a pipeline of the same size and capacity as Alternative 1 and Alternative 2. The footnote (1) should have been referenced by all three alternatives, not limited to Alternative 3. For Alternatives 1, 2 and 3 in Table 3-1, a bypass would be necessary for work requiring an isolation of the Fraser Gate IP pipeline to repair. A bypass is needed in these alternatives to allow supply from Fraser Gate to bypass the isolated section to support the portion of the system demand that an NPS 20 Coquitlam Gate IP pipeline operating at 1200 kPa does not have the capacity to deliver.

3.7. What is the NPV of costs associated with providing the anticipated bypasses over the life of the pipe.

Response:

As the NPS 20 IP pipeline in Alternative 3 would be new, FEI would not expect a significant amount of integrity related work (for e.g. valve replacements or corrosion repairs) that would require temporary bypass installations on the Coquitlam Gate IP pipeline. Seasonal maintenance flexibility would be available so planned maintenance and repair work would be scheduled within these windows negating the need for bypasses. Temporary bypasses would only be required for road lowerings and pipe relocations that cannot be scheduled at a time where sufficient maintenance flexibility is available.

However, the new NPS 20 Coquitlam IP pipeline would not improve maintenance flexibility on the existing NPS 30 Fraser Gate IP pipeline and there would still be a need for temporary bypasses to accommodate all maintenance and repair work on most of the segments of this pipeline at all times of the year. This would include integrity related work, road lowerings and pipe relocations. Although not currently forecast in FEI's long-term capital plans, it is likely that over time the NPS 30 Fraser Gate IP pipeline mainline valves will require replacement for integrity reasons. Bypasses will be required for these valve replacements. Please also see BCUC IRs 1.3.5 and 1.3.6 and CEC 1.22.1.1.

FEI is not able to estimate the number of temporary bypasses that may be required over the life of the pipeline as the majority of these bypasses would be required for unplanned work or work that is not controlled by FEI. The cost of a bypass depends on several factors including the length and diameter of the bypass, as well as routing challenges for the bypass piping. A typical NPS 20 bypass would cost approximately $0.6 million and an NPS 30 bypass approximately $0.8 million. Longer and/or larger diameter bypasses would increase this cost.

Therefore, FEI is unable to provide the NPV of costs over the life of the pipe; however, to be responsive, FEI provides the following example of approximate costs associated with a bypass.

For a capital cost of $0.6 million in 2014, the present value of the incremental revenue requirements for 60 years would be approximately $0.6 million; with a negligible impact on the Levelized Rate - $0.0002 / GJ. For a capital cost of $0.8 million in 2014, the present value of the incremental revenue requirements for 60 years would be approximately $0.8 million; still having a negligible impact on the Levelized Rate of $0.0003 / GJ.

Reference: Exhibit B-1/Section 3.2.2/Table 3-2/pg. 43

3.8. We are unable to locate the derivation of Table 3-2 in the evidence or (public) appendices. If the derivation of the table is provided in evidence please provide the reference. If not please provide the derivation of the table showing all assumptions and how the PV value is calculated.

Response:

The details supporting Table 3-2 for Alternative 6 are provided in Confidential Appendices E-3-1 and E-1-1. Annual Gross O&M is provided in the fully functional electronic spreadsheet which was filed as a confidential attachment to Confidential Appendix E-1-1 (Exhibit B-1-2).

Total Direct Capital Cost excl. AFUDC & includes Abandonment / Demolition (2014 $millions)	201.282	Confidential Appendix E-3-1, Row 11
Total Direct Capital Cost excl. AFUDC (As-spent $millions)	232.985	Confidential Appendix E-3-1, Row 11
AFUDC (As-spent $millions)	12.572	Confidential Appendix E-3-1, Row 12
Total As-spent includes Abandonment / Demolition & AFUDC ($millions)	245.557	Confidential Appendix E-3-1, Row 13
Annual incremental gross O&M (2014 $millions)	0.055	CEC IR 1.33.5
Levelized Rate Impact – 60 Yr. ($ / GJ)	0.101	Confidential Appendix E-1-1, Schedule 10, Line 39
PV Incremental Cost of Service – 60 Yr. ($millions)	300.513	Confidential Appendix E-1-1, Schedule 10, Line 22

In response to BCUC Confidential IR 1.7.3, FEI has filed the fully functional spreadsheets that contain Schedule 10 showing the Levelized Rate Impact and the PV Incremental Cost of Service for 60 years for Alternatives 4 and 5.

3.9. Please recalculate the Table 3-2 using 50 year lives.

Response:

The Levelized Rate Impact and the PV Incremental Cost of Service for 50 year financial analysis period is given in the following Table. Please note that using 50 years for the financial analysis only causes the values in the bottom two rows of Table 3-2 to change.

	Alternative 4 Install NPS 24 pipeline at 2070 kPa	Alternative 5 Install NPS 36 pipeline at 1200 kPa	Alternative 6 Install NPS 36 pipeline at 2070 kPa
Levelized Rate Impact – 50 Yr. ($ / GJ)	0.089	0.105	0.103
PV Incremental Cost of Service – 50 Yr. ($millions)	257,545	303,827	297,975

Changing the time period from 60 years to 50 years has an immaterial effect on the Levelized Rate Impact and on the present value (PV) of the incremental cost of service and does not affect the preference of Alternative 6. The effect on the Levelized Rate Impact from 60 years to 50 years for each alternative is an increase of $0.002 per GJ. For Alternative 4 this is a reduction in the cost of service PV of $2.114 million, Alternative 5 of $2.653 million and Alternative 6 of $2.538 million.

3.10. Please provide the current asset life of new NPS 20 /30 pipe. Please also provide the current depreciation rate used for this asset type.

Response:

The current depreciation rate for distribution mains is 1.55% which equates to approximately 60 years.

Neither FEI nor the pipeline industry has defined a normal range of physical service life for pipelines and as such the expected physical life for operations is evaluated for each asset individually. Based on analysis to date, no other complete IP or TP pipelines appear to require corrosion-related replacement within FEI's capital planning forecasts.

Please note that consistent with Generally Accepted Regulatory Principles, all assets including distribution mains are included in pools and depreciated based on the average remaining service life of the pool of assets. The depreciation rate is determined by a comprehensive depreciation study which takes into consideration the expected remaining service life of the individual assets in the pool.

3.11. What is the remaining net book value of the Coquitlam Gate IP pipeline?

Response:

Please refer to the response to BCUC IR 1.25.1.

Reference: Exhibit B-1, Table 3-2: Coquitlam Gate IP Project Financial Comparison, p. 43

3.12. Please provide the financial figures listed in Table 3-2 for Alternative 3.

Response:

Replacing the existing NPS 20 in kind (Alternative 3) does not restore any of the operational flexibility or system resiliency that has eroded over time as a result of customer and demand growth; therefore, FEI has deemed this alternative to be not appropriate or viable and has not included it as an alternative in the financial analysis. Further, no financial analysis was undertaken for Alternative 3 beyond calculating the capital cost in current 2014$, as such there are no equivalent figures available as summarized in Table 3-2.

Reference: Exhibit B-1/Section 3.2.3.2/Table 3-3/pg.44

3.13 Please recalculate Table 3-3 using 50 year lives.

Response:

Table 3-3 restated for a 50 year financial analysis period is provided in the table below.

	Updated Alternative 4 Install NPS 24 pipeline at 2070 kPa	Alternative 4 Install NPS 24 pipeline at 2070 kPa	Alternative 6 Install NSP 30 pipeline at 2070 kPa
Operational Risk Reduction (%)	14.34%	0	100
Remaining Operational Risk (2014 $millions / year)	2.104	2.456	0
PV Remaining Operational Risk – 50 Yr ($millions)	32.526 [1]	37.967	0
PV Incremental Cost of Service – 50 Yr ($millions)	257.545	257.545	297.975
PV Remaining Operational Risk + PV Incremental Cost of Service – 50 Yr ($millions)	290.071	295.512	297.975

The formula and the discount rate remains unchanged from the footnote 18 on page 44 of the Application; however, for the calculation of the PV Remaining Operational Risk n = 50 years.

1) Please refer to the response to BCUC IR 1.22.7 which identifies a change to the operational risk reduction under Alternative 4.(Updated Alternative 4 in table above for 50 years).

Reference: Exhibit B-1, Table 3-3: Coquitlam Gate IP Project Financial and Operational Risk Comparison, p.44; Exhibit B-2, Workshop Materials, Value of System Resiliency, p. 22

3.14 Please explain if the operational risk figure of approximately $2.5 million risk/per year includes societal/economic costs of unplanned outages.

Response:

HJ Ruitenbeek Resource Consulting Ltd. provides the following response:

The value of the operational risk is based on the "Economic Consequence Analysis" prepared in Appendix A-5. That report describes the costs included within the consequences, and categorizes them generally as: (i) direct fixed expenditures; (ii) relight costs; (iii) revenue loss to FEI; and (iv) service disruption costs to customers. All of these are regarded as "economic" costs. Categories (i), (ii) and (iii) are all directly incurred by FEI. The service disruption costs in (iv) potentially include a broad range of social adjustment costs. As described in Appendix A-5, the methodology provides an estimate of the amount businesses and individuals would need to be *theoretically* compensated to suffer a curtailment of service in gas supply; this compensation is not actually paid, but it provides a proxy for the economic and social adjustments that are needed (switching to higher price fuels, undertaking other expenditures, or just doing without certain amenities).

The figure of $2.5 million risk/per year can thus be interpreted as a probability-weighted economic consequence that includes both the private direct costs to the Company and the social and economic costs to the customers.

3.15 Please provide an operational risk figure that only considers costs to the Company in determining risk/per year of unplanned outages.

Response:

HJ Ruitenbeek Resource Consulting Ltd. provides the following response:

As noted in the response to BCOAPO IR 1.3.14, the Company costs represent 3 of the 4 cost components, notably: (i) direct fixed expenditures; (ii) relight costs; (iii) revenue loss to FEI. The operational risk will generally depend on the scenario chosen, but a good general approximation can be made by considering the relative contribution of the company costs in these three categories. Through inspection of Table 3.1 and Table 3.2 in Appendix A-5, it can be shown that the company proportion is up to 55% for the largest outages, and generally just less than 50% for residual outages caused by incidents in the Metro system sections. Therefore an estimate of a reasonable operational risk figure that includes just the company costs would be of the order of one-half of the reference amount. Based on the reference amount of $2.5 million/year operational risk presented in BCOAPO IR No. 1.3.14, this corresponds to an adjusted "Company only" amount of operational risk of the order of $1.25 million/year.

3.16 Using the operational risk figure calculated for question 3.15 above please provide a revised Table 3-3.

Response:

The following table restates Table 3-3 from the application except for the revised operational risk figure of $1.25 million per year calculated for BCOAPO IR 1.3.15, and the revised operational risk reduction for Alternative 4 as outlined in BCUC IR 1.22.7.

Revised Table 3-3: Operational Risk Reduction Excluding Societal Impact

	Alternative 4 Install NPS 24 pipeline at 2070 kPa	**Alternative 6 Install NSP 30 pipeline at 2070 kPa**
Potential Operational Risk Reduction based on response to BCOAPO IR 1.3.15	1.250	1.250
Operational Risk Reduction (Coquitlam Gate IP Pipeline complete)(NPS 30) (2014 $millions/year)		1.250
Operational Risk Reduction (Coquitlam Gate IP Pipeline complete)(NPS 24) see BCUC IR 1.22.07 (2014 $millions/year)	50% 0f 0.352=0.176	
Remaining Operational Risk (2014 $millions / year) (line 1 - line 3)	1.074	0
PV Remaining Operational Risk – 60 Yr ($millions)	17.001	0
PV Incremental Cost of Service – 60 Yr ($millions)	259.659	300.513
PV Remaining Operational Risk + PV Incremental Cost of Service – 60 Yr ($millions)	276.660	300.513

The formula, discount rate and number of years remain unchanged from the footnote 18 on page 44 of the Application Exhibit B-1.

Project Description

Reference: Exhibit B-1/Section 3.3.3.6/pg.70

3.17. Please identify the location of the 70m of required right-of-way (ROW). What is the current status of acquiring this ROW? Is the property in question subject to expropriation?

Response:

Please refer to the response to BCUC IR 1.21.1 which has been filed confidentially.

Reference: Exhibit B-1/Section 3.3.4.7/pg.80

3.18. At the Workshop of February 3, 2015 FEI noted that it had not yet finalized the routing for the Coquitlam Gate Project. This includes sections 6 and 7 as noted in Table 3-11, but also the evidence states, a revaluation of sections 5 and 6 to follow Lougheed Highway. Please explain how these various route options impact the cost of the project. In order to understand the magnitude of the potential change in routing related costs please provide the high/low estimate dependent upon the optimum and least optimum final route selection.

Response:

FEI is further evaluating Lougheed Highway as a potential route option for route corridor sections 5 and 6 only. FEI has selected the preferred route option for section 7 as described in the Application. Please also refer to the response to BCUC IR 1.18.2.

Reference: Exhibit B-1/Section 3.3.4.6 & Appendix A17/pg.53

3.19. Table 3-11 shows that the proposed routing would follow the existing NPS 20 line except for a deviation beginning at Springer Avenue (section 6) and following East 1st rather than the existing East 2nd route (section 7). Would the selection of either NPS 20 or NPS 24 pipe affect the decision with respect to deviations from the current pipeline route (i.e. allow routing along the whole of the existing pipeline)?

Response:

No, the selection of a NPS 20 or NPS 24 pipe would not affect the decision with respect to deviations from the current pipeline route.

3.20. At the Workshop of February 3, 2015 FEI noted that it had not yet finalized the routing for the Coquitlam Gate Project. Please provide a road map showing the current alternative routes that are under investigation. Please indicate when FEI believes it will resolve the issues and select its final route.

Response:

FEI is currently analyzing Lougheed Highway as a potential route option for the Coquitlam Gate IP Project in route corridor sections 5 and 6. An evidentiary update is expected to be filed in late April 2015, which will present the analysis and findings, including the route option(s) evaluated along Lougheed Highway.

Please also refer to the response to BCUC IR 1.18.2.

Reference Exhibit B-1/Section 3.3.5.4.3

3.21. Please identify any major public facilities (e.g., schools, hospitals, senior citizen facilities, etc.) that lie along the proposed routing. For any such facilities please explain what steps are being put in place to ensure pedestrian and public transit is not disrupted for these communities.

Response:

Following is a listing of some major public facilities of which FEI is aware. The list is not intended to be comprehensive and will be reviewed and updated during the detailed project planning.

VANCOUVER: Woodlands to Boundary along 1st Avenue

- Pacific Grace MB Church
- Renfrew Baptist Church
- Vancouver Mandarin Church
- Grandview Calvary Baptist Church
- Chua Chan Quang Temple
- Chilton Park
- Rupert Park Pitch & Putt
- Akali Singh Sikh Society
- Renfrew Care Centre
- Grandview Elementary School
- Chief Maquinna Elementary

BURNABY: Along Lougheed

- Gilmore SkyTrain Station
- Brentwood Town Centre SkyTrain Station
- Holdom SkyTrain Station
- Sperling-Burnaby Lake SkyTrain Station

BURNABY: Broadway Corridor

- Burnaby Mountain Golf Club
- Forest Grove Park
- Burnaby Mountain Urban Trail
- Production Way SkyTrain Station

COQUITLAM: Como Lake from North Road to Mariner Way

- Banting Secondary School
- Miller Park Elementary
- Queen of All Saints Catholic School
- Coquitlam Presbyterian Church
- Mundy Park
- Hillcrest Middle School

- Charles Best Secondary
- Coquitlam Alliance Church/Daycare
- Ranch Park Elementary School
- Parkland Elementary School

A detailed mitigation plan to address the specific construction impacts at each location has not yet been produced. This will be developed in conjunction with further route design to finalize an exact pipeline alignment. A key aspect of this effort will also involve identification and mitigation of impacts to institutional access, emergency response routes, emergency services mobilization and pedestrian and public transit. The development of Project plans to implement appropriate mitigation measures will involve ongoing consultation with affected municipalities, major Stakeholders and local residents, transit operators, and businesses, and will minimize disruptions to the communities as much as possible. Examples of possible measures to reduce the impacts to accesses, pedestrian and public transit include tailored construction staging, construction scheduling and timing, temporary rerouting of bicycle lanes and bus routes including temporary relocation of bus stops, coupled with appropriate signage, messaging and early warning and notification.

3.22. Please identify any separated bicycle lanes which will be disrupted along the proposed routing.

Response:

As there are a number of cycle routes along the proposed routing, please refer to Exhibit B-1, Appendix A-18 for a detailed listing, along with the proposed mitigation strategies.

Project Cost Estimates

Reference Exhibit B-1/Section 3.4.1.4.2

3.23. Please provide the Statistics Canada escalation rates for each year of 2002 through 2013.

Response:

Please refer to the response to BCUC IR 1.49.1 for the escalation rates from 2002 through 2014.

4. Fraser Gate IP

Reference: Exhibit B-1/Section 4.3.3.2.2/pg.113

4.1 Please provide the seismic vulnerabilities report that was used to support the 1997 modifications to the Fraser Gate Station.

<u>Response:</u>

Please refer to Attachment 4.1.

4.2 Please explain FEI's understanding of why the pipe routing risk was not identified as part of the 1997 project.

<u>Response:</u>

Please refer to the response to CEC IR 1.52.7.1.

5. Project Costs and Accounting

Reference: Exhibit B-1/Section 5.2

5.1. Please explain how/when FEI expects to adjust rate base and associated revenue requirement for this project.

Response:

Pursuant to the 2014-2019 PBR Decision, CPCNs will be held as work in progress until the beginning of the year after the project has gone into service. Both the Coquitlam Gate IP Project and the Fraser Gate IP Project are scheduled to be in service in 2018, therefore the capital costs and impact on revenue requirements and delivery rates will occur on January 1, 2019 when the Project costs are transferred to Gas Plant in Service accounts.

As described in Section 5.2.2 and 5.2.3 of the Application, the project development and application cost deferral accounts are proposed to enter rate base on January 1, 2016 with amortization expense to commence in 2016. As such, these project-related costs will impact the revenue requirement and will be recovered through delivery rates effective January 1, 2016 through to December 31, 2018.

5.2. Please explain what steps are being introduced to ensure the project costs are at or below the forecasts provided to the Commission in this application.

Response:

As noted in the Application, for the Coquitlam Gate IP Project, the estimated capital cost of $232.985 million plus actual AFUDC (As-spent) will be used as the control budget. Cost reports when required will conform at a minimum to the level of detail as set out in Confidential Appendix E-3-1.

The expected accuracy range (+30/-20%) of the cost estimate is $322.175 million to $199.397 million.

For the Fraser Gate IP Project, the estimated capital cost of $17.231 million plus actual AFUDC (As-spent) will be used as the control budget. Cost reports when required will conform at a minimum to the level of detail as set out in Confidential Appendix E-3-2.

The expected accuracy range (+30/-20%) of the cost estimate is $23.639 million to $14.586 million.

Project management best practices will be utilized throughout the lifecycle of the Projects. The control budget will provide the baseline reference for subsequent project monitoring and control and assessment of financial performance during the Projects. Project controls will be put in place where processes and tools will be used to manage and mitigate potential cost issues and any risk events that may impact the Projects' costs. These project controls will provide the means to recognize variances from the cost management plans.

FEI notes the IR above implies that the actual Projects' costs should be "[…] *at or below the forecasts provided to the Commission in this application*". While FEI will use all reasonable efforts to ensure that Projects' costs are minimized where possible, the Company considers that all costs prudently incurred in carrying out the Projects are recoverable from ratepayers.

FEI believes that periodic reports of costs as incurred or anticipated to be incurred throughout the Projects are sufficient to address higher than projected costs, should they become a concern. To that end, should the Commission provide direction to do so, and as indicated in the Draft Order included as Appendix G-2 of the Application, FEI will file with the Commission quarterly progress reports on the Projects. The Quarterly Progress Reports will address in some detail the risks that the Projects are experiencing, the options available to address the risks, the actions that FEI is taking to deal with the risks and the likely impact on Projects' schedule and cost.

5.3. Please explain why provisions are expected to be included in the project tendering that will aid in keeping the project within budget, what these provisions are expected to be, and why FEI believes they will aid in keeping the project within budget.

Response:

A detailed contracting strategy will be developed and selected incorporating 'lessons learned' from previous major projects with similar magnitude and scope. The tender(s) will be structured with sufficient consideration of the specific Project conditions which will avoid conflicts between the contracting parties. The strategy will provide for appropriate allocation of risk between FEI and the contractor(s) such that claims, disputes and other undesirable results will be minimized.

The Projects will have specific challenges such as traffic management, the existence of third party utilities and unanticipated sub-surface conditions, access and restricted workspace difficulties and market conditions at the time of bidding. FEI will review these conditions to determine the optimum balance between transfer of risks to ensure cost certainty and cost minimization. To increase the probability of project success, the contract(s) will be clear on responsibility splits, assignments of various scopes of work to all project participants (owner, contractors, and suppliers) and be aware of and address scope of work interfaces.

5.4. The project materials and contractor selection are not expected to be completed for another 2 years. Please explain if any deferral or variance accounting is being sought to capture cost changes/overruns for this project that may occur prior to the start of construction.

Response:

FEI is not applying for a deferral account to capture variances in the materials or construction costs for the Projects. All actual material and construction costs will be properly charged to the respective plant asset accounts in accordance with GAAP and the Uniform System of Accounts for Gas Companies.

5.5. FEI has stated that it will not complete materials tendering and ordering until August 2016 and awarding of a contract almost a year later (June 2017). The project is not expected to start until mid-2018. In light of the lengthy timelines and current uncertainties as to routing: Please comment on the following proposed conditions for the CPCN:

- That the certificate contain certain project time milestones which if not met FEI would need to reapply or seek an adjustment to its certificate
- That the certificate contains conditions related to an approved budget and FEI would need to reapply or seek an adjustment to its certificate if FEI were to materially deviate from this budget.

Response:

In light of the planning, permitting, designing, procuring, and constructing requirements for a Project of this magnitude, FEI does not consider the timeline to be lengthy or unusual. Additionally, assuming that the comment about "current uncertainties as to routing" is referring to the potential alignment along the Lougheed Highway, as stated in the response to BCUC IR 1.15.1, the Company will provide an evidentiary update during this proceeding when the Company completes its evaluation. The Commission will have all of the information that it requires to make an informed decision regarding the Application.

FEI believes that the suggested conditions are not appropriate.

With respect to the first proposed condition, FEI believes that the need for these Projects is established based on the evidence. They will have to be implemented regardless of the specific timing of construction, thereby making this type of condition of no value. Put another way, a construction delay will not change the need for the Projects. The Company has expressed a clear desire to complete the Projects in a reasonable amount of time, but must retain the flexibility to manage the Projects appropriately. The Commission retains oversight of the execution of the Projects, and FEI has the responsibility for ongoing management of the execution of the Projects and will report regularly.

The second condition suggested is also unnecessary and inappropriate. The costs and benefits of a proposed project are forecast on the best information available at the time of application for a CPCN. The Projects are required, and the cost of the Projects do not change that need such that it would be appropriate to revoke or revisit a CPCN if the budget or cost were to change. It is, of course, appropriate to expect that the Company will execute the Projects prudently. The Commission has the ability to oversee the progress of the Projects and has tools available to examine costs incurred after the fact.

Consistent with other CPCN projects of the Company, FEI anticipates providing some form of periodic reports to the Commission and also considers a requirement for reporting of significant delays or material cost variances to be appropriate. Such reporting requirements strike an appropriate balance between the Commissions' oversight of the execution of the Projects and the Company's responsibility for the ongoing management of the Projects.

Reference: Exhibit B-1/Section 9.3/Table 9.4

5.6. Given that FEI has not finalized its routing what routing does it propose to be included in the CPCN?

Response:

Please refer to the response to BCUC IR 1.15.1.

6. Reference: Exhibit B-1, Archaeological Overview Assessment, p. 145; and First Nations Consultation, p. 180

Preamble: "Potential archaeological and cultural impacts associated with the four areas of high archaeological potential will be further assessed during the AIA, which will be undertaken once approval of this Application from the Commission is obtained and prior to construction."

"The potential of the Projects to impact First Nations interests is confined to impacts on archaeological sites (if any) from construction activities associated with the pipeline upgrades"

6.1. How long will a detailed AIA take to complete?

Response:

The AIA will be conducted in advance of construction and will be coordinated with the overall project schedule. The time required to complete the AIA will depend upon the size of the assessed area of high archaeological potential and the detailed construction design. Therefore, until a detailed construction design is determined, only an estimated timeline can be provided. Based on the information available now, it is estimated that the AIA will take approximately one week of field time and about 3 days of reporting time. However, this time frame may change significantly once detailed designs are complete and if any archaeological artifacts are uncovered during the AIA.

6.2. Please explain why a detailed AIA will only be done "once approval of this Application from the Commission is obtained" and consequently will not be provided to the Commission or the intervenors during this proceeding.

Response:

Projects of this magnitude typically follow a staged planning and permitting process. When this staged planning and permitting process is applied to archaeological work, this means that an Archaeological Overview Assessment is undertaken to determine if areas of archaeological potential exist within the project footprint. If there are areas of archaeological potential that may be impacted by the project, then a Heritage Inspection Permit is required to undertake the AIA.

Once approval from the Commission is obtained, the Projects will proceed into detailed engineering design and a detailed construction design will be prepared. An AIA will be undertaken in areas where there is a potential for the detailed construction design to interact with archaeological resources. Until such time as the detailed design is known, it is not cost prudent to undertake an AIA because the footprint of the potential impact is not known.

7. Reference: Exhibit B-1, First Nations Consultation, Engagement Activities, pp.177-178

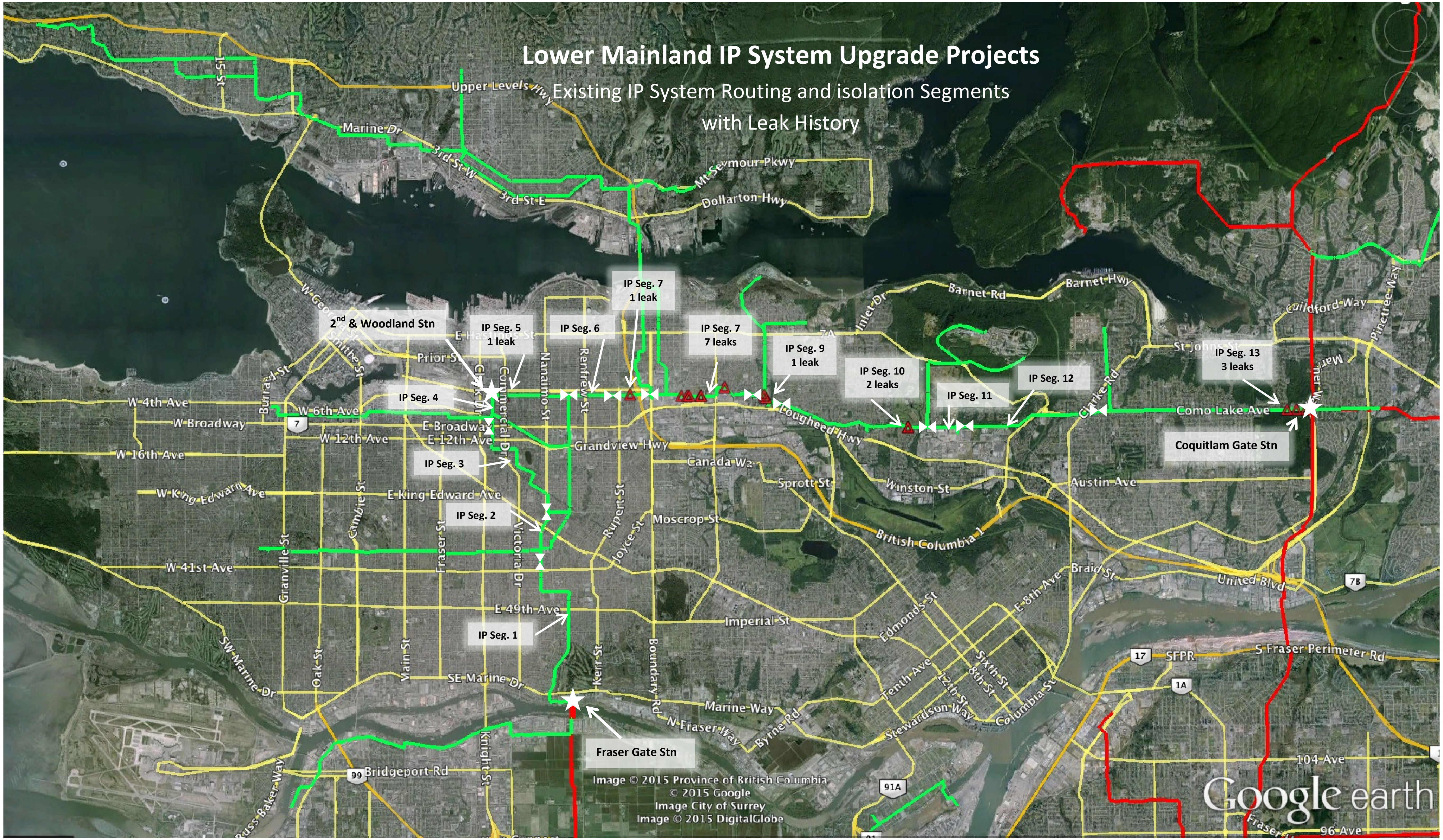
Preamble: A number of First Nations who were contacted by mail by the Company have not responded. The Company has said that “no significant concerns have been raised as of December 4, 2014.” The Company has also said that they will be providing notification about the proceedings at the BCUC to the identified First Nations.

7.1 Does FEI plan to follow up with the First Nations who have not responded to find out their position on the proposed projects? If so, please describe what this follow up will consist of.

Response:

FEI has engaged First Nations by providing them with information regarding the Projects and by inviting their questions and further involvement. The Company believes that a First Nation will respond if it is interested in receiving more information, or participating in the review of the Projects. FEI is not planning to pursue the First Nations that have not indicated they are interested in the Projects further, but will engage with those interested in knowing more about the Projects.

Attachment 1.4


Lower Mainland IP System Upgrade Projects
Existing IP System Routing and isolation Segments
with Leak History
2nd & Woodland Stn
IP Seg. 5
1 leak
IP Seg. 6
IP Seg. 7
1 leak
IP Seg. 7
7 leaks
IP Seg. 9
1 leak
IP Seg. 10
2 leaks
IP Seg. 11
IP Seg. 12
IP Seg. 13
3 leaks
Coquitlam Gate Stn
IP Seg. 4
IP Seg. 3
IP Seg. 2
IP Seg. 1
Fraser Gate Stn
Upper Levels Hwy
Marine Dr
3rd St W
3rd St E
Mt Seymour Pkwy
Dollarton Hwy
Barnet Rd
Barnet Hwy
Guildford Way
Pinetree Way
St Johns St
Como Lake Ave
Clarke Rd
Inlet Dr
Lougheed Hwy
W Georgia St
Smithe St
Burrard St
Prior St
E Hastings St
W 4th Ave
W 6th Ave
W Broadway
W 12th Ave
W 16th Ave
E Broadway
E 12th Ave
W King Edward Ave
E King Edward Ave
W 41st Ave
E 49th Ave
SE Marine Dr
SW Marine Dr
Granville St
Cambie St
Oak St
Main St
Fraser St
Victoria Dr
Commercial Dr
Clark Dr
Nanaimo St
Renfrew St
Rupert St
Joyce St
Kerr St
Boundary Rd
Grandview Hwy
Canada Way
Sprott St
Winston St
Moscrop St
British Columbia 1
Imperial St
Marine Way
N Fraser Way
Byrne Rd
Edmonds St
Tenth Ave
12th St
8th St
Sixth St
Stewardson Way
Columbia St
E 8th Ave
Braid St
Austin Ave
United Blvd
S Fraser Perimeter Rd
SFPR
104 Ave
96 Ave
Bridgeport Rd
Russ Baker Way
Knight St
15 St
7
7A
17
1A
7B
91A
99
Image © 2015 Province of British Columbia
© 2015 Google
Image City of Surrey
Image © 2015 DigitalGlobe
Google earth

Attachment 3.1

From:	Balmer, Bryan
Sent:	Tuesday, February 18, 2014 4:32 PM
To:	'Kevin.Parsonage@bcogc.ca'
Cc:	Recsky, Keith
Subject:	FortisBC update on General Order 2013-25 -- January 2014 update

Hi Kevin,

Thank-you for the response to FortisBC's submission to General Order 2013-25, pertaining to the 508 mm pipeline installed in 1958 as Pipeline Project 1045 ("subject pipeline"). I will provide monthly updates on FortisBC's weekly leak detection surveys, as well as an updated timeline regarding the application to the Commission for replacement of the subject pipeline.

Update for January 2014:
- Weekly leak surveys of the subject pipeline were completed.
- No leaks were identified on the subject pipeline.
- The expected filing date of the OCG application to replace the subject pipeline is Q2 2015.

Sincerely,
Bryan Balmer

Bryan Balmer, P.Eng.
Manager, System Integrity Programs
FortisBC
Direct: 604-592-7701
Cell: 604-908-3060
www.fortisbc.com

From:	Balmer, Bryan
Sent:	Tuesday, March 25, 2014 2:06 PM
To:	Kevin.Parsonage@bcogc.ca
Cc:	Recsky, Keith
Subject:	FortisBC update on General Order 2013-25 -- February 2014 update

Hi Kevin – please find below FortisBC's update for February 2014 on General Order 2013-25, which pertains to a 508 mm pipeline installed in 1958 as Pipeline Project 1045 ("subject pipeline"):

- Weekly leak surveys of the subject pipeline were completed.
- Leak survey identified a potential leak on the subject pipeline on February 26, 2014. The leak was confirmed on the subject pipeline on March 3, 2014, and reported to the OGC on this date (DGIR # 133610).
- The expected filing date of the OCG application to replace the subject pipeline is Q2 2015.

Sincerely,
Bryan Balmer

Bryan Balmer, P.Eng.
Manager, System Integrity Programs
FortisBC
Direct: 604-592-7701
Cell: 604-908-3060
www.fortisbc.com

From:	Balmer, Bryan
Sent:	Thursday, April 24, 2014 8:48 AM
To:	Kevin.Parsonage@bcogc.ca
Cc:	Recsky, Keith
Subject:	FortisBC update on General Order 2013-25 -- March 2014 update

Hi Kevin – please find below FortisBC's update for March 2014 on General Order 2013-25, which pertains to a 508 mm pipeline installed in 1958 as Pipeline Project 1045 ("subject pipeline"):

- Weekly leak surveys of the subject pipeline were completed.
- Leak survey identified a potential leak on the subject pipeline on February 26, 2014. The leak was confirmed on the subject pipeline on March 3, 2014, and reported to the OGC on this date (DGIR # 133610).
- The expected filing date of the OCG application to replace the subject pipeline is Q2 2015.

Sincerely,
Bryan Balmer

Bryan Balmer, P.Eng.
Manager, System Integrity Programs
FortisBC
Direct: 604-592-7701
Cell: 604-908-3060
www.fortisbc.com

From: Balmer, Bryan
Sent: Thursday, May 8, 2014 12:47 PM
To: Kevin.Parsonage@bcogc.ca
Cc: Recsky, Keith
Subject: FortisBC update on General Order 2013-25 -- April 2014 update

Hi Kevin – please find below FortisBC's update for April 2014 on General Order 2013-25, which pertains to a 508 mm pipeline installed in 1958 as Pipeline Project 1045 ("subject pipeline"):

- Weekly leak surveys of the subject pipeline were completed.
- No leaks were identified on the subject pipeline.
- The expected filing date of the OCG application to replace the subject pipeline is Q2 2015.

Sincerely,
Bryan Balmer

Bryan Balmer, P.Eng.
Manager, System Integrity Programs
FortisBC
Direct: 604-592-7701
Cell: 604-908-3060
www.fortisbc.com

From: Balmer, Bryan
Sent: Wednesday, July 9, 2014 3:44 PM
To: Kevin.Parsonage@bcogc.ca
Cc: Recsky, Keith
Subject: FortisBC update on General Order 2013-25 -- May 2014 update

Hi Kevin – please find below FortisBC's update for May 2014 on General Order 2013-25, which pertains to a 508 mm pipeline installed in 1958 as Pipeline Project 1045 ("subject pipeline"):

- Weekly leak surveys of the subject pipeline were completed.
- No leaks were identified on the subject pipeline.
- The expected filing date of the OCG application to replace the subject pipeline is Q2 2015.

Sincerely,
Bryan Balmer

Bryan Balmer, P.Eng.
Manager, System Integrity Programs
FortisBC
Direct: 604-592-7701
Cell: 604-908-3060
www.fortisbc.com

From:	Balmer, Bryan
Sent:	Wednesday, July 9, 2014 3:44 PM
To:	Kevin.Parsonage@bcogc.ca
Cc:	Recsky, Keith
Subject:	FortisBC update on General Order 2013-25 -- June 2014 update

Hi Kevin – please find below FortisBC's update for June 2014 on General Order 2013-25, which pertains to a 508 mm pipeline installed in 1958 as Pipeline Project 1045 ("subject pipeline"):

- Weekly leak surveys of the subject pipeline were completed.
- No leaks were identified on the subject pipeline.
- The expected filing date of the OCG application to replace the subject pipeline is Q2 2015.

Sincerely,
Bryan Balmer

Bryan Balmer, P.Eng.
Manager, System Integrity Programs
FortisBC
Direct: 604-592-7701
Cell: 604-908-3060
www.fortisbc.com

From:	Balmer, Bryan
Sent:	Wednesday, September 24, 2014 4:24 PM
To:	Kevin.Parsonage@bcogc.ca
Cc:	Recsky, Keith
Subject:	FortisBC update on General Order 2013-25 -- July 2014 update

Hi Kevin – I appreciate your note, and please accept my apologies for the delayed reporting.

Please find below FortisBC's update for July 2014 on General Order 2013-25, which pertains to a 508 mm pipeline installed in 1958 as Pipeline Project 1045 ("subject pipeline"):

- Weekly leak surveys of the subject pipeline were completed.
- No leaks were identified on the subject pipeline.
- The expected filing date of the OCG application to replace the subject pipeline is Q2 2015, although this may vary depending on the timing of required project approvals from the BC Utilities Commission.

The paperwork for a portion of the August leak survey has not yet been finalized in our work management system, so I will follow-up on that with our Operations group. In the interim, I can report that we have not identified any leaks on the subject pipeline in August or September to date.

Sincerely,
Bryan Balmer

Bryan Balmer, P.Eng.
Manager, System Integrity Programs
FortisBC
Direct: 604-592-7701
Cell: 604-908-3060
www.fortisbc.com

From:	Balmer, Bryan
Sent:	Friday, October 17, 2014 3:08 PM
To:	Kevin.Parsonage@bcogc.ca
Cc:	Recsky, Keith
Subject:	FortisBC update on General Order 2013-25 -- August 2014 update

Hi Kevin – Please find below FortisBC's update for August 2014 on General Order 2013-25, which pertains to a 508 mm pipeline installed in 1958 as Pipeline Project 1045 ("subject pipeline"):

- Weekly leak surveys of the subject pipeline were completed.
- No leaks were identified on the subject pipeline.
- FortisBC is expecting to be filing an application to the BC Utilities Commission in Q4 2014 for replacement of the subject pipeline. This application will contain estimated dates for a number of key project milestones. As the BCUC application preparation has not yet been fully completed, I am unable to provide the full details at this time. However, I would appreciate a phone call at your convenience to discuss next steps for communication between FortisBC and the OGC on this matter.

Sincerely,
Bryan Balmer

Bryan Balmer, P.Eng.
Manager, System Integrity Programs
FortisBC
Direct: 604-592-7701
Cell: 604-908-3060

From:	Balmer, Bryan
Sent:	Thursday, December 18, 2014 3:15 PM
To:	Kevin.Parsonage@bcogc.ca
Cc:	Recsky, Keith; Kobialko, Cari; Kilpatrick, Melanie
Subject:	FortisBC update on General Order 2013-25 -- September to November 2014 update

Hi Kevin – Please find below FortisBC's update for September, October, and November 2014 on General Order 2013-25, which pertains to a 508 mm pipeline installed in 1958 as Pipeline Project 1045 ("subject pipeline"):

- Weekly leak surveys of the subject pipeline were completed.
- No leaks were identified on the subject pipeline.
- FortisBC is expecting to be filing an application to the BC Utilities Commission in Q4 2014 for replacement of the subject pipeline. The application is expected to contain the following estimated dates for various schedule milestones:

Coquitlam Gate IP Project - Schedule Milestones

Activity	Date
Concept Development	Completed
CPCN Preparation	July 2013 – Dec. 2014
CPCN Filing	Dec. 2014
CPCN Approval	Q3 2015
Start Detailed Engineering, material specification and contract development	Oct. 2015
Materials Tendering and Orders Placed	Aug. 2016
Award Contractor	June 2017
Submit OGC Application	Sept. 2017
OGC Pipeline Approval	Jan. 2018
Materials Delivery	Mar. 2018
Construction Start	April 2018
In Service	Nov. 2018
Restoration	June 2019

Please let me know if you would like to coordinate a meeting between FortisBC and the OGC to review our application to the BC Utilities Commission.

Sincerely,
Bryan Balmer

Bryan Balmer, P.Eng.
Manager, System Integrity Programs
FortisBC
Direct: 604-592-7701
Cell: 604-908-3060

From:	Balmer, Bryan
Sent:	Friday, January 23, 2015 11:00 AM
To:	Kevin.Parsonage@bcogc.ca
Cc:	Recsky, Keith
Subject:	FortisBC update on General Order 2013-25 -- December 2014 update

Hi Kevin – Please find below FortisBC's update for December 2014 on General Order 2013-25, which pertains to a 508 mm pipeline installed in 1958 as Pipeline Project 1045 ("subject pipeline"):

- Weekly leak surveys of the subject pipeline were completed.
- No leaks were identified on the subject pipeline.
- FortisBC has filed an application to the BC Utilities Commission for replacement of the subject pipeline (dated December 19, 2014). The application contains the following estimated dates for various schedule milestones:

Note: this is an excerpt from Section 3.3.6, page 90 of Application – which can be accessed from http://www.bcuc.com/ApplicationView.aspx?ApplicationId=476, scroll down to "B" Exhibits, select B-1.

Table 3-12: Coquitlam Gate IP Project Schedule Milestones

Activity	Date
Concept Development	Completed
CPCN Preparation	July 2013 – Dec. 2014
CPCN Filing	Dec. 2014
CPCN Approval	Q3 2015
Start Detailed Engineering, material specification and contract development	Oct. 2015
Materials Tendering and Orders Placed	Aug. 2016
Award Contractor	June 2017
Submit OGC Application	Sept. 2017
OGC Pipeline Approval	Jan. 2018
Materials Delivery	Mar. 2018
Construction Start	April 2018
In Service	Nov. 2018
Restoration	June 2019

Sincerely,
Bryan Balmer

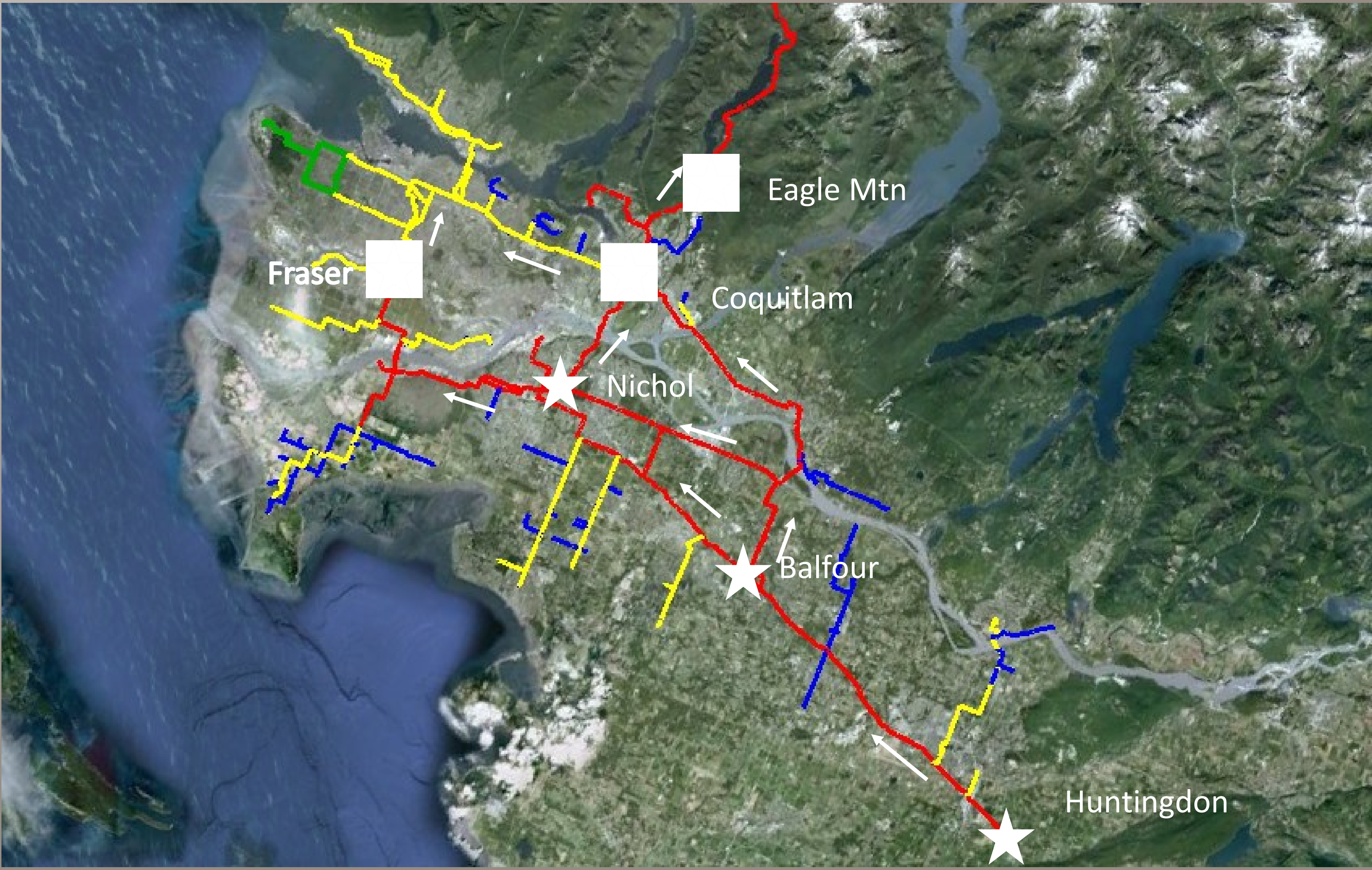
Bryan Balmer, P.Eng.
Manager, System Integrity Programs
FortisBC
Direct: 604-592-7701
Cell: 604-908-3060

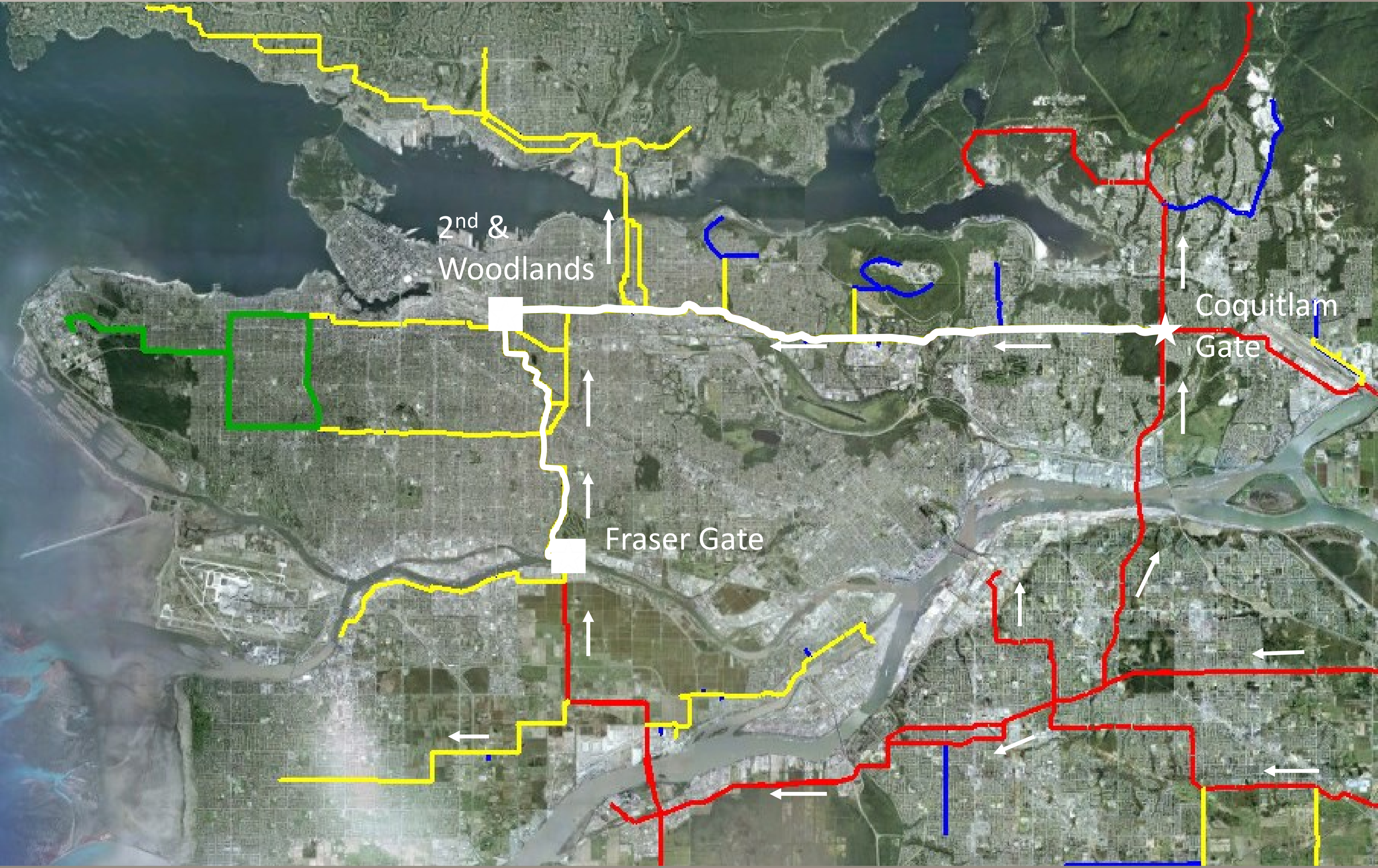
508mm (20”) IP Coquitlam to Burnaby IP

OGC Kelowna office

October 15, 2013

FORTIS BC™


Eagle Mtn
Fraser
Coquitlam
Nichol
Balfour
Huntingdon


2nd &
Woodlands
Coquitlam
Gate
Fraser Gate

BACKGROUND

- *Installed in 1959*
- *Cathodic protection*
- *Part of Lower Mainland TP/IP “grid”*
- *Plant applied coating: coal tar*
- *Field applied coating at girth welds: coal tar*
- *41,000 customers directly connected*
- *Operating at 1200 kPa (175 psig)*
- *17% SMYS*
- *Sweet, dry natural gas*
- *Urban environment (roads, residences, malls, etc.)*
- *Public right of way*

FAILURE MECHANISM

- ***External corrosion***
- ***Disbonded field applied coating at girth welds***
  - poor adhesion
  - 4 to 8 o’clock position
  - thicker coating at bottom
- ***Coating shields CP***

LEAK HISTORY

- ***Leaks due to external corrosion***
  - 1987: 1
  - 1994: 1
  - 1999: 1
  - 2001: 1
  - 2010: 1
  - 2011: 1
  - 2012: 3
  - 2013 (YTD): 6

ANALYSIS

- ***Records***
  - Unable to pinpoint weld locations
- ***Above ground coating surveys***
  - Shielding issues
- ***In Line Inspection***
  - Gas powered tools not practical
    - Launcher / receiver location limitations
    - Multiple stations
    - Obstructions (elbows, boiler plugs, etc.)
  - Tethered tool access issues
  - Small, concentrated corrosion features difficult to determine depth

SHORT TERM STRATEGY

- ***Consequence management***
  - ***Leak survey***: increased frequency to weekly
  - ***Odorization***: continue existing
- ***OGC incident reporting requirements???***

LONG TERM STRATEGY

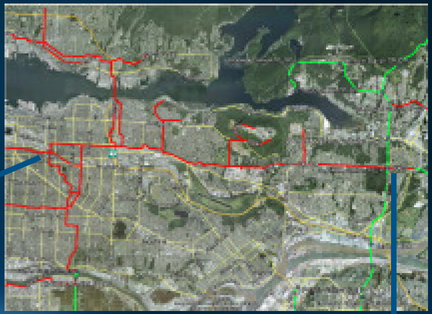
- ***Part of reliability / capacity strategy for Lower Mainland***
- ***Determine if other "similar" pipelines***
- ***Submission to BCUC in 2014 (?)***
- ***Earliest replacement timetable is 2016 (?)***
- ***Communication with stakeholders***
  - OGC
  - Municipalities
  - Residents, businesses, public, etc

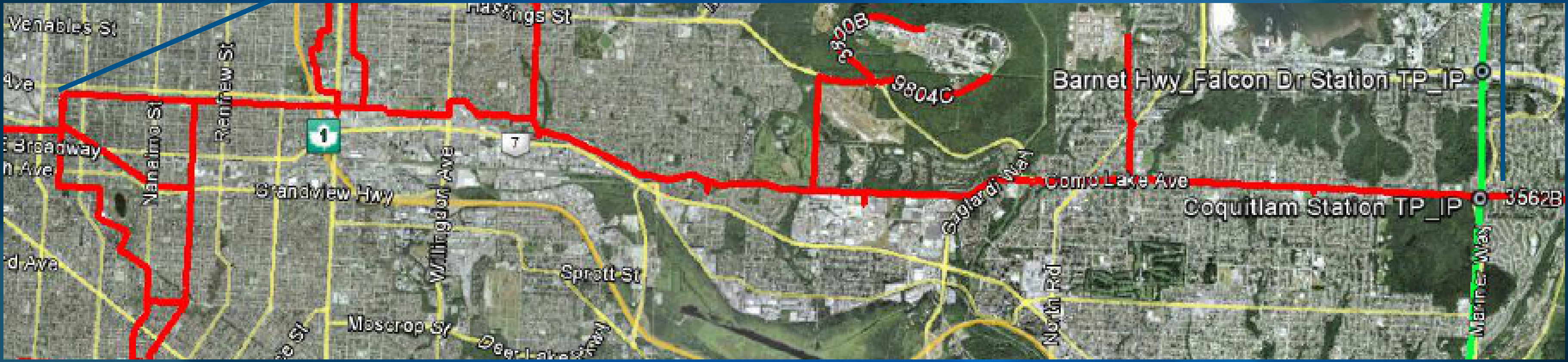
508mm Coquitlam IP Pipeline

Asset Management Plan:

Progress Review/Discussion Meeting

27th March 2013





FORTIS BC™

Asset Management Strategy

1. Provide an engineering based summary of the pipeline design, operation and on-going integrity program;
2. Identify any risks to the pipeline system;
3. Review the options available to FortisBC to manage the pipeline going forward; and
4. Prepare and present this information in a format that will enable/facilitate short and long-term asset management decision-making for this pipeline.

Mar '13

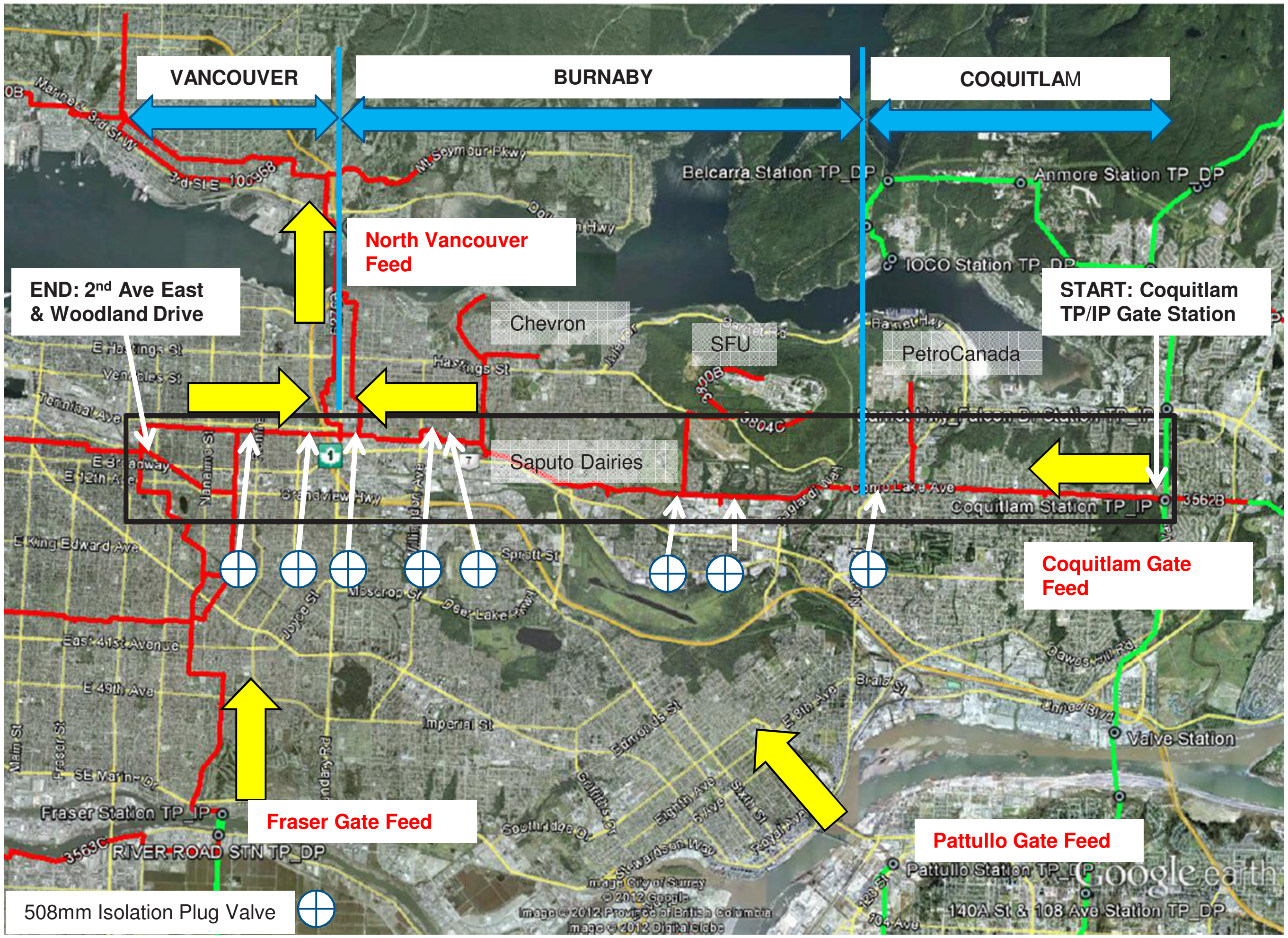
- (NOTE: LTSP program – 508mm Coquitlam IP pipeline solution decision in 2013, and project delivery commencing thereafter ?)

Basis of Design

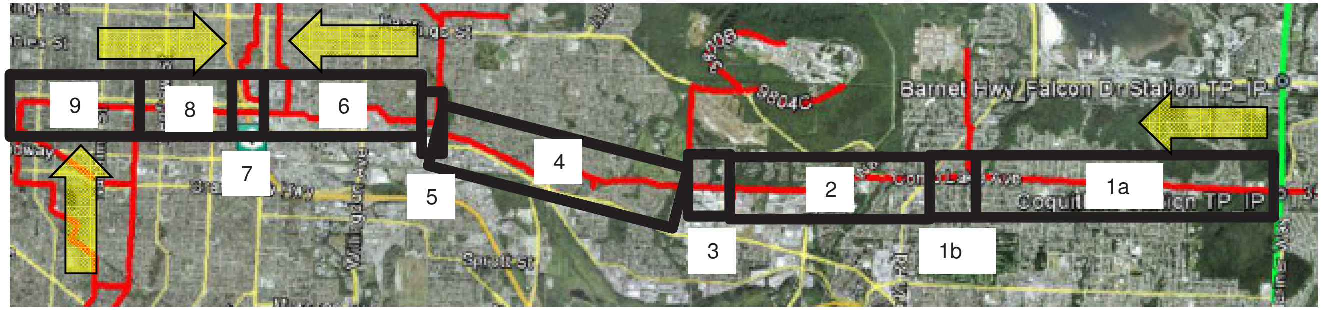
Details	Values
Asset Name	508mm Coqutlam - Vancouver IP Pipeline
Original Construction Year	1958
Years In Service	54
Pipeline Diameter	508 mm (NPS 20)
Pipeline Length	Approx. 19 km
Pipeline Buried Depth	3’-6” to 4’-0” nominal
Pipeline Material Grade/SMYS	API 5L - X42/290 MPa
Class Location	4
Pipeline Wall Thickness (mm)	6.34 mm
Hydro-Test Data	1,827 kPag
Maximum Operating Pressure (MOP)	1,200 kPag
Operating Temperature Range	0-7°
External Factory Coating	Coal Tar Enamel
External Field Coating	Coal Tar Enamel
Repair Coating	Cold Applied Polymer Tape
Internal Lining	None
Corrosion Allownace	Impressed Current Cathodic Protection
Conventional In-Line Inspection	Not Feasible

FORTIS BC™

Operation

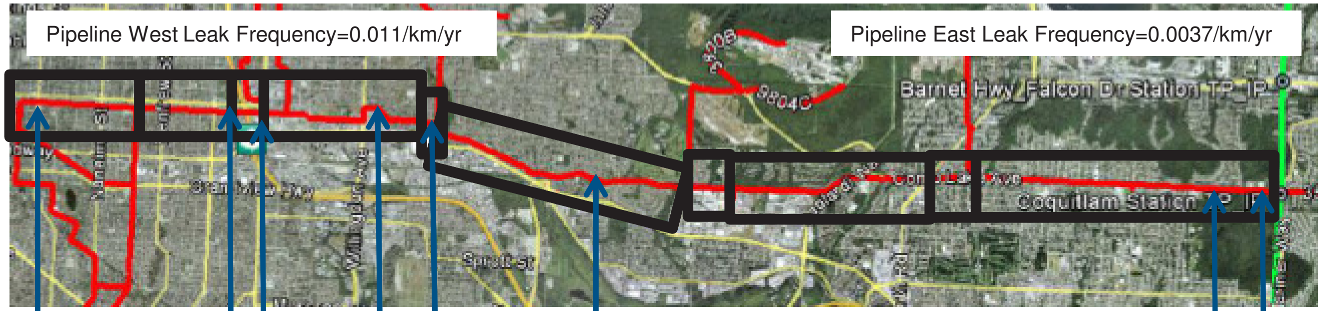

VANCOUVER
BURNABY
COQUITLAM
North Vancouver Feed
END: 2nd Ave East & Woodland Drive
START: Coquitlam TP/IP Gate Station
Chevron
SFU
PetroCanada
Saputo Dairies
Coquitlam Gate Feed
Fraser Gate Feed
Pattullo Gate Feed
508mm Isolation Plug Valve
Belcarra Station TP_DP
Anmore Station TP_DP
IOCO Station TP_DP
Coquitlam Station TP_IP
Fraser Station TP_IP
RIVER ROAD STN TP_DP
Pattullo Station TP_IP
140A St & 108 Ave Station TP_DP
Valve Station
Google earth

Shut-In Analysis



Segment	Degree Day	Start Month	End Month	Location	Comments
1a)	18	April 1st	Oct 31st	Como Lake Ave and Westwood Dr. to Como Lake Ave and Robinson	New valve or Stopple Fitting at Robinson and Como Lake Ave east of Robinson. Maintains feed to Petro can from West. Some district stations isolated
1b)	18	April 1st	Oct 31st	Como Lake and Robinson St. to Como Lake Ave and Clark Rd.	feed to Petro Can is maintained through a bypass
2	18	April 1st	Oct 31st	Como Lake Ave and Clarke Rd to Broadway and Underhill Ave	
3	18	April 1st	Oct 31st	Broadway and Underhill Ave to Broadway and Arden Ave.	
4	18	April 1st	Oct 31st	Broadway and Arden Ave to Broadway and Springer Ave	Saputo load has to be maintained with LNG Bainbridge and Lougheed station is isolated
5	18	April 1st	Oct 31st	Broadway and Springer Ave to Springer Ave and Halifax St	
6	18	April 1st	Oct 31st	Springer Ave and Halifax St to 2nd Ave and Boundary Rd	
7	18	April 1st	Oct 31st	2nd Ave and Boundary Rd to E 2nd Ave and Cassiar	NPS 20IP on Boundary Rd to 2nd Narrows bridge is isolated as is Kootenay and Dundas St district station
8	18	April 1st	Oct 31st	E 2nd Ave and Cassiar top E 2nd Ave and Slocan St	
9	31	All year	All year	E 2nd Ave and Slocan St to E 2nd Ave and Woodland Drive	This section can be taken out of service at any time of year

Recorded Leak Locations



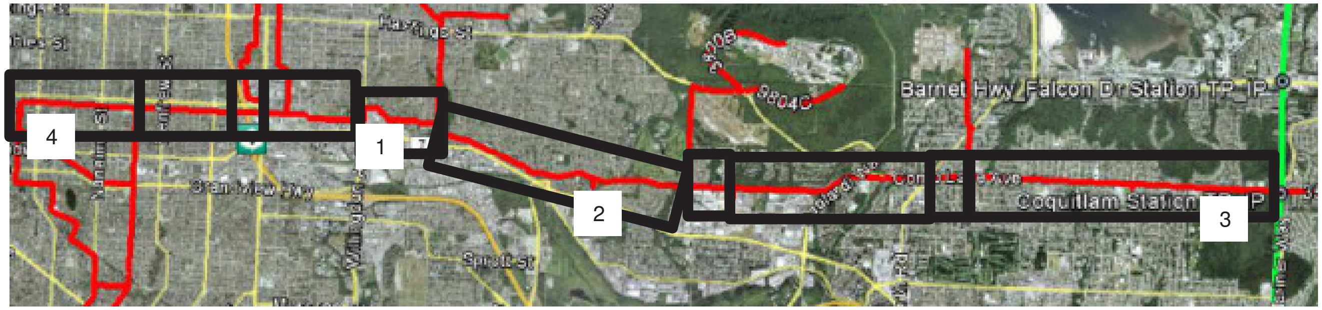
Pipeline West Leak Frequency=0.011/km/yr

Pipeline East Leak Frequency=0.0037/km/yr

Date	Location
2012/05/24	2525 Como Lake Avenue (weld)
2012/04/30	3422 E. 2nd Avenue (weld)
2011/03/06	7584 Broadway (weld)
2010/02/18	Como Lake Ave. @ Mariner Way (weld)
2001/02/21	Brentlawn Ave. @ Fairlawn Ave. (weld)
1999/08/18	3434 E. 2nd Ave (weld)
1994/11/07	E. 2nd Ave. @ Commercial Dr. (weld)
1987/11/18	Springer Ave. @ Braelawn (pipe)

- 24 corrosion leaks recorded on the FortisBC IP pipeline system province wide
- Of these, 8 (33%) have occurred on the 20” Coquitlam IP pipeline
- The 8 leaks are not unique in terms of location, geology or hydrology

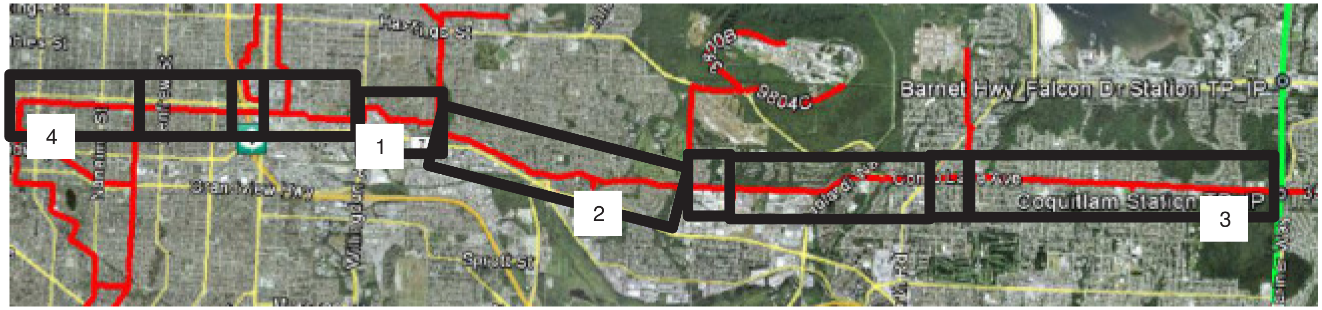
Integrity Management



Site Location	Direct examinations	Indirect indications exposed	Corrosion features detected	Corrosion features undetected by indirect assessment
Site 1- Brentlawn Ave	3	3	4	4
Site 2- Broadway Ave	3	1	1	1
Site 3- Como Lake Ave	3	5	13	8
Site 4- East 2nd Ave	4	5	10	5
Total	13	14	28	18

- Inspection program started Oct 2011 in the vicinity of 4 recorded leak sites
- To better understand the causes of the leaks
- To assess pipeline and coating condition and validate above ground survey techniques
- In total 100m of pipeline exposed

Integrity Inspection Results



Site Location	Corrosion features	Active corrosion features	Corrosion beneath field-applied coating	Corrosion beneath factory-applied coating
Site 1-Brentlawn Ave	4	4	4	0
Site 2-Broadway Ave	1	1	1	0
Site 3-Como Lake Ave	13	13	7	6
Site 4-East 2nd Ave	10	4	7	3
Total	28	22	19	9

- Of 28 corrosion features 22 had active corrosion, most corrosion located beneath field applied coating; therefore:
- Possibility of additional undetected/active corrosion at welds (event likelihood)
- Potentially leading to through wall corrosion (event consequence)
- Resulting in future gas leaks (risk)

Pipeline Risks & Impacts

- **Integrity:**
- Undetected coating damage – may cause further corrosion
- Active corrosion – may lead to further leaks
- **Operational:**
- Security of supply - future leaks may cause loss of customer supply
- Reliability (confidence in performance) - unscheduled outages to facilitate repair would negatively impact the pipeline reliability
- Redundancy/Backup – limited; dependent upon system capacity and isolation valve configuration
- Maintenance – proactive leak detection and reactive leak repair may become more onerous in the future
- **Safety:**
- Leak hazard – gas migration from gas leak

Pipeline Risk/Impact Mitigation Options

		O&M	CAPEX
Proactive Integrity Analysis			
1	2012 IP Line Management Plan	$ 1 M/yr	
2	Enhanced Direct/Indirect Inspection	$ 2 M/yr	
Rehabilitate Existing Pipeline (same running line)			
3	Modify 508mm isolation valves		$ 1 M
4	Locate and replace girth welds		$ 1 M/km
5	Segment replacement (in-kind)		$ 3.8 M/km
6	Complete replacement (in-kind)		$ 75 M
7	Complete replacement with larger pipeline (30”/42”)		$ 95 / 140 M
New Pipeline (new running line)			
8	New looped pipeline parallel to existing pipeline (24”/30”/42”)		$ 67 / 85 / 125 M
9	New pipeline route (via Indian Arm) (24”/30”/42”)		$ 100 / 125 / 180 M

FORTIS BC™

Options - Evaluation

- Pipeline options should mitigate some/all of the risks
- Analysis (subjective):
  - Each option assigned a ‘benefit score’ in terms of its ability to mitigate each risk
  - Each option assigned a ‘project delivery’ score in terms of project delivery
  - Overall Option score = Sum [benefit score x project delivery score]

	Overall Score (Project Benefit x Project Delivery)		
Maximum (score = 0.12)	0.12	0.36	0.72
Medium (score = 0.7)	0.07	0.21	0.42
Minimum (score = 0.1)	0.01	0.03	0.06
Pipeline Option Benefit (weighting 2.0) Pipeline Option Project Delivery (weighting 1.0)	High (Cost and Risk) (score = 0.1)	Medium (Cost and Risk) (score = 0.3)	Low (Cost and Risk) (score = 0.6)

Options – Comparison & Selection

	Enhanced Integrity Management		Existing Pipeline Replacement (same running line)			New Pipeline (new running line)	
	2012 IP Line Management Plan	Enhanced Integrity Analysis	Locate/Replace Girth Welds	Segment Replacement (in-kind)	Complete Replacement	New Loop	New Pipeline
INTEGRITY							
Mitigate Active Corrosion	??	??	??	??	??	??	??
Mitigate Leaks	??	??	??	??	??	??	??
OPERATION							
Ensure Supply Security	??	??	??	??	??	??	??
Enhance Reliability	??	??	??	??	??	??	??
Provide Redundancy	??	??	??	??	??	??	??
Minimise Maintenance	??	??	??	??	??	??	??
SAFETY							
Eliminate Leak Hazards	??	??	??	??	??	??	??
Total	??	??	??	??	??	??	??
Rank	**??**	**??**	**??**	**??**	**??**	**??**	**??**

FORTIS BC™

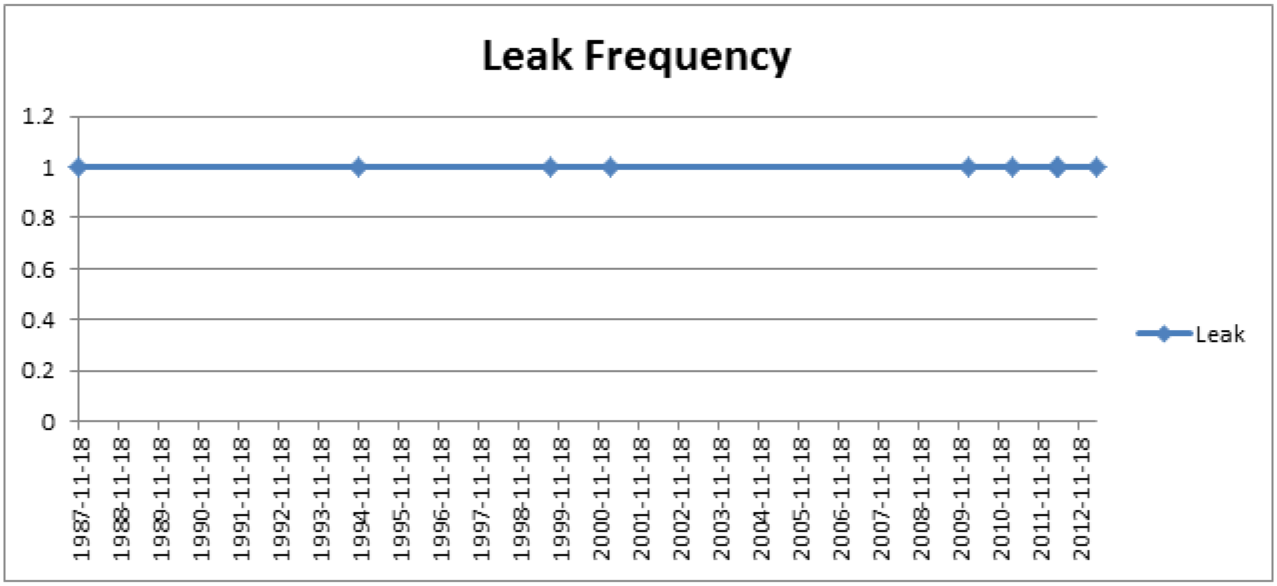
Management Strategy – Next Steps

1. Complete further direct/indirect inspections to develop integrity baseline for entire pipeline.
2. Finalise System Capacity study; confirm pipeline operational flexibilities.
3. Any other drivers? (Financial? Overall LML system constraints?)
4. Confirm that pipeline options list is inclusive, practical and deliverable (FEED, estimates, major stakeholder pre-consultation etc.)
5. Use Step 1, 2, 3 and 4 to complete the pipeline ptions analysis, selection and recommendation.
6. Use all of the above to inform the final Asset Management Strategy

July 5, 2013 Update on NPS 20” Coquitlam Gate IP

Further to our meeting of June 19 and the request for further information specific to the Coquitlam IP pipeline condition and the reasoning behind our position that it is nearing the end of its service life:

A review of AM/FM shows nine leaks on the pipeline since 1987 with an increase in frequency, the space between the data points on the graph below illustrates that frequency.



Leak Frequency on the Coquitlam IP Pipeline

During the drafting of this email, another potential leak is being investigated in Burnaby that is not illustrated in the graph above.

While the instance of leaks increasing may not be a completely reliable means of projecting future failures, it does agree with the findings of the digs and examinations completed by System Integrity. In 2012, the group examined 14 welds; nine showed evidence of corrosion and six showed evidence of active corrosion.

When combining the increasing frequency of leaks and the observations made during the integrity digs we can conclude confidently that the pipeline will continue to develop leaks. This conclusion was supported by an independent analysis completed by Dynamic Risk in 2013**. The 100% probability of future failures leaves FortisBC only two options:**

- **put procedures in place to stop the corrosion; or,**
- **replace the pipe.**

Coating and cathodic protection are the two industry accepted means of limiting or preventing corrosion. In the case of the Coquitlam IP pipeline, examination has shown that the coating has failed at a number of locations. The coating has become disbonded from the pipe and shields the pipe from cathodic protection; this actually accelerates the corrosion. As a result of the shielding nature of the coating used at the girth welds where the majority of corrosion occurs, increasing the energy levels of the cathodic protection system would have little result. **With instances of coating failure and in view of the shielding nature of the coating, no further feasible options are available to stop the corrosion.**

There are a number of factors that make it inadvisable or contrary to Code requirements for a company to continue to operate a pipeline with a history of leaks. The primary factor that all companies must consider is safety. Migration of gas tends to follow the path of least resistance and it is common for gas leaking from a pipeline to migrate into foreign utilities or structures. In a worst case scenario this could lead to significant property damage and/or injury and/or loss of life. A first hand example of the risks associated with gas migration from a leak occurred in Quesnel in April, 1997 when gas accumulation in a commercial premise from a nearby leak was associated with six fatalities. The Coquitlam IP pipeline runs in lanes, under streets and near other utilities in suburban and urban areas. **With the hard cover over disturbed soil offering migration paths, the possibility of leak migration and accumulation is unacceptably high.**

Code requirements prohibit a company from operating a pipeline that has too many leaks.

CSA Z662, *Oil and gas pipeline systems* is one of the governing Codes for the Coquitlam IP pipeline. Clause 12.10.2.3 is applicable to the pipeline and states:

> (c) Pipe containing leaks that can create a hazard shall be repaired as specified in Clause 12.10.6 or 12.10.9, and such repairs shall be documented. Leaks located by leakage surveys shall be investigated promptly, and any necessary repairs shall be made and documented.
> (d) Where the condition of distribution or service lines, as indicated by leak records or visual observation, deteriorates to the point where they should not be retained in service, they shall be replaced, reconditioned, or abandoned.

The Coquitlam IP pipeline has met the threshold described in Clause 12.10.2.3 (d) and must be replaced.

The *Oil and Gas Activities Act (OGAA)* defines spillage

> **"spillage"** means petroleum, natural gas, oil, solids or other substances escaping, leaking or spilling from
> > (a) a pipeline, well, shot hole, flow line, or facility, or
> > (b) any source apparently associated with any of those substances.

Clause 37 of the *OGAA* goes on to require:

> (1) A permit holder and a person carrying out an oil and gas activity must
> > (a) prevent spillage, and
> > (b) promptly report to the commission any damage or malfunction likely to cause spillage that could be a risk to public safety or the environment.
>
> (2) If spillage occurs, a permit holder or person carrying out an oil and gas activity must promptly do all of the following:
> > (a) remedy the cause or source of the spillage;
> > (b) contain and eliminate the spillage;
> > (c) remediate any land or body of water affected by the spillage;
> > (d) if the spillage is a risk to public safety or the environment, report to the commission
> > > (i) the location and severity of the spillage, and
> > > (ii) any damage or malfunction causing or contributing to the spillage.
>
> (3) A person who is aware that spillage is occurring or likely to occur must make reasonable efforts to prevent or assist in containing or preventing the spillage.

The only effective way to prevent spillage is to replace the pipeline.

Clause 53 of the *OGAA* also details:

> Control of oil and gas activities
>
> 53 (1) If, in the commissioner's opinion,
>
> (a) a permit holder has engaged in a pattern of conduct that shows that the person is unfit to carry out the oil and gas activities permitted by the permit holder's permit, and
>
> (b) there is a risk to public safety, the environment or petroleum and natural gas resources, the commission may
>
> (c) enter, seize and take control of any well, pipeline, facility or storage reservoir together with any associated chattel and fixture and any pertinent records,
>
> (d) either discontinue all activity or take over the management and control of the well, pipeline, facility or storage reservoir,
>
> (e) take the steps the commission considers necessary
>
> (i) to prevent the flow or release of petroleum, natural gas or other substances from any stratum that a well enters, including plugging a well at any depth, or
>
> (ii) for public safety or to protect the environment, and
>
> (f) carry out any other prescribed actions.
>
> (2) If the commission takes control of a well, pipeline, facility or storage reservoir,
>
> (a) the commission may issue orders concerning the well, pipeline, facility or storage reservoir to
>
> (i) the permit holder, and
>
> (ii) an officer, employee, agent and contractor of the permit holder operating the well, pipeline, facility or storage reservoir,
>
> and, if the commission issues an order to a person referred to in either subparagraph (i) or (ii), the order applies to both the person referred to in subparagraph (i) and the persons referred to in subparagraph (ii), and
>
> (b) subject to section 55, the commission may take, deal with and dispose of all petroleum, natural gas or other substances from the well, pipeline, facility or storage reservoir.
>
> (3) The commissioner may order by whom and to what extent costs and expenses incurred as a result of proceedings taken under this section are to be paid.

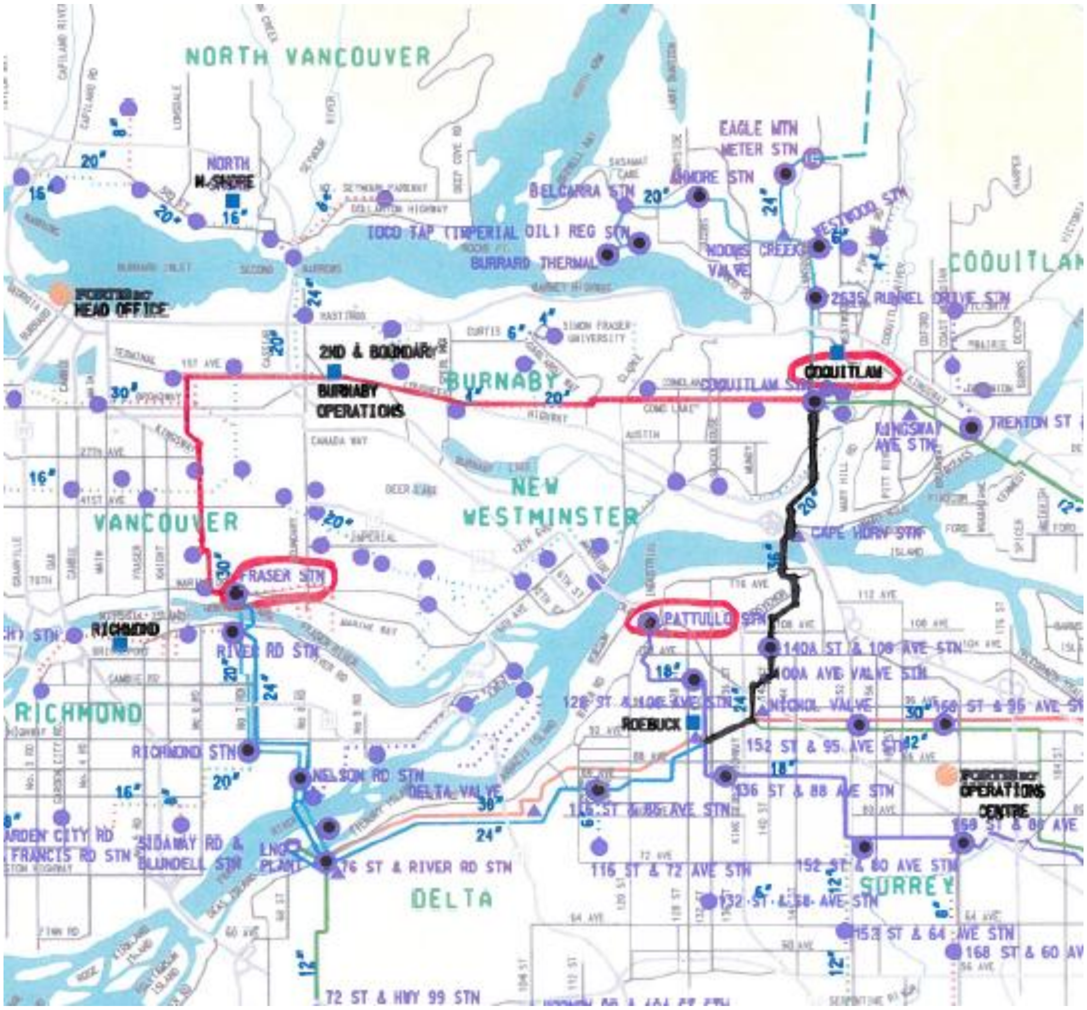
Clause 53 provides wide reaching power and, in extreme consequences **the Commission could force a course of action on FortisBC .**

While repairs of some of the leaks experienced to date on the Coquitlam IP have cost in excess of $100k, the operating cost has not yet reached the point where, on its own, it warrants pipeline replacement. This; however, is only one of the factors to be considered. In addition to the risks to safety and repair costs, disruption to the public and society must be considered. Closing a major transportation route during repairs or forcing the evacuation of a business can have significant societal consequences. While no impacts have been quantified, the Coquitlam IP pipeline does run adjacent to schools, power lines, churches, shopping malls, arterial roads and highways, as well as Skytrain tracks and stations. Significant incidents associated with the Coquitlam IP pipeline could disrupt any of these facilities with the associated impacts to the public.

The **potential consequences to the public and the economic impact as a result of disrupting businesses and/or transportation facilities support replacement of the pipeline**.

Deferring the replacement of the pipeline will result in more than simply requiring FortisBC to continue to respond to an increasing number of leaks. Adopting a position of deferring the work will result in FortisBC being in non-compliance with CSA Z662 and the Oil and Gas Activities Act (and potentially other Acts and Laws), increasing risk to the public and exposing the company to intervention by the Oil and Gas Commission.

The Coquitlam IP pipeline must be replaced as a high priority.



Metro Vancouver Natural Gas Supply

The majority of the gas supply to the Metro Vancouver area is delivered through Fraser Gate Station on the south side of the City of Vancouver. Due to the current system configuration, there is no time that the outlet of Fraser Station can be shut off without losing service to more than 100,000 customers. The other sources of supply to the area; Coquitlam Station and IP Pipeline, and Patullo Station do not have adequate capacity to take over supplying the area in the event of loss of service from Fraser Station.

In the 1990s work was done on the transmission pipelines supplying gas to Fraser Station and to the station itself to provide increased resistance to earthquakes; no work was done on the IP outlet of Fraser Station. A review by Honegger and Associates identified the Fraser IP pipeline to Marine & Elliott Station as vulnerable to a seismic event as low as 1:475; the National Building Code requirement is for 1:2475. FortisBC has adopted the 1:2475 seismic design criteria and has justified projects and design requirements to this threshold; for example, a significant portion of the justification of the replacement of the South Arm of the Fraser River was based on meeting the 1:2475 seismic criteria.

The Fraser IP pipeline does not meet current seismic criteria and must be replaced.

As noted above, there is insufficient capacity through Coquitlam and Patullo to enable shutting in the Fraser IP. This provides **FortisBC two options for strengthening the Fraser IP pipeline:**

- **Install a temporary bypass during construction, or**
- **Increase capacity through Coquitlam and/or Patullo station to provide adequate supply to enable shutting in the Fraser IP**.

The Fraser IP pipeline runs through a heavily suburban area with multiple townhouses and includes a crossing of the rail line that runs parallel to the Fraser River. Any temporary bypass would need to be installed on the surface of the ground and would need to be raised above the rail line to the degree than trains could pass under it. As a result, a bypass would be vulnerable to vandalism, vehicle traffic and rail traffic damage during construction of the strengthened IP pipeline. Even if the public concerns could be managed, a pipeline of the size and pressure that would be required for a bypass would present an unacceptable level of risk.

A temporary bypass from Fraser Station to Marine & Elliott is not feasible.

Patullo Station supplies natural gas to New Westminster through a 690 kPa distribution pressure system that does not have sufficient capacity to provide a back feed to the Fraser IP pipeline; **increasing the capacity though Patullo Station is not feasible**.

The remaining option is to increase the capacity through Coquitlam IP to provide adequate supply to enable replacing the Fraser IP with a pipeline of sufficient design strength to resist a 1:2475 earthquake.

As noted above, the Coquitlam IP pipeline must be replaced as a high priority to address increasing integrity risks. **When the Coquitlam IP is replaced, it must be replaced with sufficient capacity to enable replacing the Fraser IP pipeline**.

Deferral of replacing the Fraser IP will mean that FortisBC will continue to operate with a critical pipeline that has been identified as not meeting the company’s design criteria. In the event of an earthquake, failure of the pipeline is probable with associated risk to the public in the immediate area. Assuming a complete pipeline failure the minimum safe distance from any ensuing fire would be 230 feet; this puts a number of town homes and the rail line at risk. Further, loss of service through the Fraser IP would result in loss of service of up to 170,000 customers.

Excerpts from Corporate Board Reports

2013 Q2 Report:

Asset sustainment planning has been an area of significant activity in the second quarter. Included in the 2014-2018 FEI RRA filed recently were known major sustainment capital projects including several which are expected to exceed the CPCN project threshold of greater than $5 million each. Preparatory work for 2014 CPCN applications was initiated including ongoing development and refinement of a long term Coastal Transmission System plan to mitigate identified integrity and system reliability risks.

2013 Q4 Report

Asset sustainment planning, including development of CPCN applications and 20-year asset plans, was advanced throughout 2013. Asset plans encompass growth, integrity and sustainment, security of supply, as well as operational requirements. The initial focus for CPCNs is on the Coastal operating area, influenced by an assessed need to replace a 508 mm (20 inch) diameter intermediate-pressure pipeline, due to corrosion, that runs from Coquitlam to Vancouver. A leak on this pipeline in the fourth quarter resulted in the Company having a single reportable incident to the BC Oil and Gas Commission. This leak, along with six prior leaks on this line in 2013, was successfully managed by Operations without significant service loss or other impacts.

2014 Q1 Report

FortisBC had one reportable incident to the BC Oil and Gas Commission in the first quarter of 2014. This was a corrosion leak on the 508 mm (20 inch) diameter intermediate-pressure pipeline that runs from Coquitlam to Vancouver. No significant service loss or other impacts resulted from the leak. A CPCN application is in progress for replacement of this pipeline due to systemic and non-preventable corrosion. The CPCN, currently planned for submission in the third quarter 2014, will also address broader issues impacting the Lower Mainland natural gas system such as growth, security of supply, and operational requirements.

2014 Q2 Report

Gas system assets continued to be operated and maintained in accordance with the FortisBC Integrity Management Program (IMP). Recent activities and achievements were as follows:

- *Numerous public information sessions were held during the second quarter in support of the planned third quarter 2014 CPCN filing for the Lower Mainland Gas System Upgrade (LMSU) program. The LMSU includes replacement of the 508 mm (20 inch) diameter intermediate pressure pipeline that runs from Coquitlam to Vancouver, which is required due to systemic and non-preventable corrosion.*

Attachment 4.1

Golder Associates Ltd.

500 - 4260 Still Creek Drive
Burnaby, British Columbia, Canada V5C 6C6
Telephone (604) 298-6623
Fax (604) 298-5253

Golder Associates

REPORT ON

SITE-SPECIFIC SEISMIC VULNERABILITY ASSESSMENT OF BC GAS FRASER GATE STATION NO. 3 VANCOUVER, B.C.

Submitted to:

BC Gas Utility Ltd.
1111 West Georgia Street
Vancouver, B.C.
V6E 4M4

DISTRIBUTION:

10 copies -	BC Gas Utility Ltd. Vancouver, B.C.
1 copy -	EQE International Inc. Irvine, California, U.S.A.
1 copy -	Douglas Honegger, Engineering Consultant Torrance, California, U.S.A.
2 copies -	Golder Associates Ltd. Burnaby, B.C.

January 20, 1997

942-1211A

NHRM DB#46

Golder Associates Ltd.

500 - 4260 Still Creek Drive
Burnaby, British Columbia, Canada V5C 6C6
Telephone (604) 298-6623
Fax (604) 298-5253

Golder Associates

January 20, 1997

942-1211A

BC Gas Utility Ltd.
3777 Lougheed Highway
Burnaby, B.C.
V5C 3Y3

Attention: Mr. Dan McGuire, P.Eng.
Manager, Special Projects

RE: SITE-SPECIFIC SEISMIC VULNERABILITY ASSESSMENT OF BC GAS FRASER GATE STATION NO. 3 VANCOUVER, B.C.

Dear Dan:

Please find enclosed ten (10) copies of our final report on the site-specific vulnerability assessment of the Fraser Gate Station.

We trust that the report is satisfactory, and that it provides the information you require in a suitable format. If you have any questions please do not hesitate to contact us.

Yours very truly,

GOLDER ASSOCIATES LTD.

D. Wijewickreme, Ph. D., P. Eng.
Senior Geotechnical Engineer

T.P. Fitzell, P. Eng.
Principal

DW/TPF/vw
942-1211A/1070

J:\RPT-97JAN\DW21211A.DOC

TABLE OF CONTENTS

EXECUTIVE SUMMARY

A site-specific seismic vulnerability assessment of Fraser Gate Station No. 3 of the BC Gas Lower Mainland Transmission system has been completed. The scope of this assessment included evaluating the seismic vulnerability of: (a) the gate station piping components, (b) the transmission pipelines (508 mm and 610 mm diameter) entering the gate station from the south, and (c) the outlet pipeline segment between the gate station and South West Marine Drive. Identification of potential mitigative measures which could be considered to reduce the risk of pipeline rupture to acceptable levels was also included within the scope of work. The response of the site and the expected performance of the pipelines under earthquake loadings corresponding to 1:100 year, 1:475 year, and 1:2,000 year seismic risk levels was investigated.

A detailed geotechnical investigation involving both on-shore and off-shore field testing as well as laboratory testing was carried out to obtain information on the soil and groundwater conditions. The results of the investigation indicate that the southern half of the gate station site, the area approximately south of the existing regulator house, including the river bank of the North Arm of the Fraser River is underlain by a saturated loose to compact sand layer extending to depths in the order of 10 to 12 m below the ground surface. In the northern part of the gate station compound, a layer of silt extending to depths in the order of 6 to 8 m below the ground surface was encountered. These soils were found to be underlain by dense granular strata.

Dense to very dense (glacial till-like) soil strata were encountered at relatively shallow depths (<1.5 m) along most parts of the pipeline alignment between South East Marine Drive and the Fraser gate station. The exception was in the area between the railway crossing and the Fraser gate station where the depth to dense strata appears to generally increase from relatively shallow depths to a depth of about 6 m at the Fraser gate station to the east. The soils above the dense strata are inferred to primarily consist of surficial fills underlain by soft silty soils similar to those encountered in the northern part of the Fraser gate station.

On completion of the geotechnical investigation, detailed analyses were carried out to assess the site performance under seismic loading. The assessment of ground motion response and slope stability and the estimation of earthquake-induced ground deformations were performed as part of these analyses.

Analysis of the response of the site to ground shaking indicates that there is a high risk of liquefaction in the southern half of the gate station compound and within the river channel. In saturated granular soils, liquefaction occurs when the shear strains induced due to ground shaking cause transient pore water pressures to increase in the soil mass and, as a result, reduce the intergranular contact stresses to negligible levels. In this transient state, the soil mass is subject to significant reduction in shear strength and behaves essentially as a viscous fluid that could deform or flow under gravitational or inertia forces.

Based on the slope stability and ground displacement analyses, liquefaction-induced lateral ground movements are identified as the principal hazard to the pipelines entering the gate station from the south and to equipment located within the station compound. For the seismic loadings corresponding to all the risk levels considered in the study, large ground displacements in excess of 3 m which would lead to a flow slide towards the river are predicted to influence an area extending up to about 30 m north from the crest of the river bank. For return periods of 1 in 475 and 1 in 2,000 years, lesser, but still significant, ground displacements are predicted to influence an area extending to about 40 m from the crest of the river bank.

For the area along the outlet pipeline, the risk of liquefaction-induced hazards under seismic loading is estimated to be low. The earthquake-induced ground surface displacements along the segment of the pipeline between the Fraser gate station and the railway crossing are not expected to be more than 0.3 m. Lesser ground movements are expected along Kent Avenue and Elliott Street north of the railway crossing.

The results of the structural vulnerability assessment carried out by EQE International Inc. indicate that the predicted earthquake-induced differential displacements at the Fraser gate station exceed the estimated capacity of the pipelines by an order of magnitude. In view of this, the only remedial measures deemed practical relate to reducing the deformations by improving the ground conditions. Provided that ground improvement can reduce the earthquake-induced permanent ground deformations to less than about 150 mm, no modification of the existing station piping is judged necessary.

The 762 mm outlet pipeline is estimated to be capable of withstanding the expected ground movements corresponding 1 in 2,000 year return period level earthquake loading. For these reasons, no soil or pipeline improvements are judged necessary for the outlet pipeline.

Given the magnitude of the maximum permanent ground deformations estimated for the gate station site, failure of the transmission pipelines entering the gate station from the south (beneath the river) is possible. Even if ground improvement is carried out at the site, there will still be a potential for large ground deformations south of the zone of ground improvement. The impact of such deformations would be to place tensile loads on the transmission lines that would be resisted by their effective anchorage within the improved soil region of the gate station. Based on the structural computations, the maximum tensile stress induced in the pipes under such ground movements is in the order of 50% of the minimum tensile yield stress of the pipe and is judged not to pose a credible threat of rupture. As such, modification of the pipeline within the gate station to account for possible soil loading occurring at the river crossing is not expected to be necessary. This conclusion, however, needs to be carefully reviewed as part of the detailed design of soil remediation measures to confirm that the pipe within the station boundary is effectively anchored.

Densification of selected soil zones within the river bank area and/or within the gate station compound would be effective in minimizing the potential ground movements under earthquake loading. Such treatment of soils should preferably be carried out to form a non-liquefiable barrier (or barriers) aligned perpendicular to the predominant direction of potential ground movements. The selection of the most suitable ground improvement technique is governed by several factors, such as soil conditions, equipment space restrictions, pipeline protection issues, environmental regulatory requirements, land availability etc. Based on our geotechnical requirements, the method of vibro-replacement (involving the installation of "stone columns") is considered to be the most suitable technique of ground densification for use at the Fraser gate station site.

Preliminary analyses indicate that the introduction of a densified barrier, likely in the order of 15 to 20 m wide, would significantly reduce the predicted large earthquake-induced ground movements in the vicinity of the gate station. The actual sizing and configuration of ground improvement zones, and selection of the most suitable technique, should be determined during detailed design, with due consideration given to the other factors identified above. Based on very preliminary analyses, the cost of vibro-replacement ground improvement for an assumed densification zone extending over a 15 m x 100 m area parallel to the river bank (i.e., extending approximately from the east end of the gate station compound to the west end), including mobilization costs, was estimated to be in the order of $200,000 in June 1995, at the time the evaluation of mitigation options was carried out. Additional costs related to ancillary work such as site preparation and restoration work, environmental controls, and engineering construction inspection etc. are expected to range between $200,000 to $250,000.

1.0 INTRODUCTION

Golder Associates Ltd. was retained as the prime engineering consultant by BC Gas Utility Ltd. (hereafter referred to as "BC Gas") to carry out a site-specific seismic vulnerability assessment of the BC Gas Fraser gate station in Vancouver, B.C. Further to a regional study on the seismic vulnerability of the BC Gas Coastal Transmission Pipeline System, carried out in 1993, the Fraser gate station and the outlet pipeline segment between the Fraser gate station and South East Marine Drive was ranked high among the locations identified as vulnerable under seismic loading. Following review of the recommendations of the regional study, detailed site-specific assessment of the site was requested by BC Gas with the objective of evaluating the vulnerability of the gate station and the outlet pipeline in detail and also identifying potential mitigative measures which could be considered to reduce the risk of pipeline rupture to acceptable levels.

The scope of work for this study is described in detail in our proposal to BC Gas dated November 7, 1994, and can be summarized as follows:

(a) Carry out geotechnical field investigations to determine the soil and groundwater conditions at the Fraser gate station site and the outlet pipeline segment between the Fraser gate station and South East Marine Drive;

(b) Evaluate the response of the site to earthquake loading from a geotechnical point of view, including liquefaction assessment, prediction of earthquake-induced ground deformations, and derivation of soil parameters as input to structural assessment;

(c) Carry out a detailed structural review of transmission and station piping to obtain a qualitative assessment of the piping vulnerability using the site-specific geotechnical data; and

(d) Identify conceptual site remediation schemes.

The detailed design and assessment of potential remedial schemes, and preparation of contract drawings and specifications for any remediation work required are not included within the current scope of work.

EQE International Inc. of Irvine, California, U.S.A. was retained as the structural engineering sub-consultant for this project.

This report presents the factual results of the geotechnical investigation, along with our assessment of the site performance and structural vulnerability of the piping and gate station components, identification of conceptual structural/geotechnical retrofit methods, and a preliminary indication of the costs involved in such remedial work.

2.0 BACKGROUND INFORMATION

2.1 1993 Regional Vulnerability Assessment of BC Gas Coastal Transmission Pipeline System

In 1993, a team of engineering consultants headed by EQE International of Irvine, California U.S.A. (hereafter referred to as "EQE") was retained by BC Gas to carry out a regional study to assess the seismic vulnerability of the BC Gas Coastal Transmission Pipeline system. Golder Associates Ltd. served as the geotechnical engineering sub-consultant for this project. The scope of the study included the seismic vulnerability assessment of transmission and large diameter (greater than NPS 8) intermediate pressure pipelines and the associated above-ground facilities of the BC Gas pipeline system. The objective of the risk assessment was to identify and prioritize portions of the system which are vulnerable under seismic loading. Given the lack of redundancy in the gas transmission pipelines, and the implications of significant interruption in supply, a very low risk level is considered desirable.

The 1993 study was regional in nature, and it did not include site-specific detailed vulnerability assessment of the system. A probabilistic approach using a seismicity model recently developed by BC Hydro International Ltd., was used in defining potential earthquake hazards.

The damage associated with the liquefaction-induced permanent ground movements was identified as the primary hazard to the pipeline system. The vulnerability of the pipeline system was assessed for seismic risk levels corresponding to return periods of 475, 1,000, and 2,000 years. The peak ground acceleration (PGA) predictions corresponding to the median attenuation relations were used in the assessment of liquefaction-induced lateral ground displacement hazard. The MLR empirical method proposed by Bartlett and Youd (1992) was used in the assessment of these ground deformations. The MLR method, being a model derived from statistical back-analysis of field observed ground movements, provided a mechanism to account for uncertainties in the median estimates of computed ground displacements. This methodology, therefore, allowed the vulnerability assessment to be carried out using median values throughout, with the uncertainties accounted for and calibrated with field observations at the end of the hazard estimate process. Moreover, since the objective of the study was to identify and rank the pipeline system in terms of vulnerability this procedure was considered appropriate for the regional study.

Segments of pipelines located within potentially liquefiable deposits, referred to as "slope groups", were identified based on an evaluation of the predicted lateral displacements. With the use of GIS database software, combined with non-linear finite element analyses of typical pipeline configurations, the cumulative probability of pipeline rupture from seismic hazards identified above were estimated for each slope group. The results were used to rank the slope groups according to risk.

As a result of the risk assessment, 49 slope groups were identified as having a risk of rupture greater than that corresponding to a 2,000 year return period. Of these 49 slope groups, the top 30 were estimated to have a risk corresponding to a return period less than 475 years. The slope groups ranked within the top ten fell into the category of 10% probability of failure in the next 20 years (i.e., 1 in 200 year return period).

Based on the regional study, the slope group between the Fraser gate station site and South East Marine Drive (including the Fraser gate station) was ranked highest in terms of vulnerability.

2.2 Site Location, Topography, and General Geology

2.2.1 Fraser Gate Station

The Fraser gate station is located in the 2700 Block of East Kent Avenue South in Vancouver, B.C., as shown in Figures 1 and 2. The gate station compound covers an approximately rectangular plan area (about 95 m long in the east-west direction and about 65 m wide in the north-south direction), and it is situated on the north bank of the North Arm of the Fraser River. The site is bounded by East Kent Avenue South to the north, part of the Riverfront Park owned by the City of Vancouver to the west, and a residential development, which is presently under construction, to the east. The southern fence of the gate station compound is located about 8 to 10 m north of the crest of the river bank. The strip of land between the crest of the river bank and the southern fence is presently used as a walkway which is understood to be a part of the Riverfront Park.

The site topography within the station compound and also in the east-west direction is generally flat. Based on the information provided to us by BC Gas (i.e., pipeline survey data drawings No. 785 and 786, titled "Plan and Profile of Existing 24 inch and 20 inch Pipeline Crossing North Arm/Fraser River at Fraser Gate", dated December 17, 1994, prepared by U.L.S.I. Inc. of Houston Texas), the river bank slopes down towards the south at slopes ranging from 1 horizontal to 1 vertical (1 V : 1 H) to 1 V: 3 H within the rip-rap area which extends to about 6 m below crest level. The river bed below this level slopes southward at an average gradient of about 8% to the horizontal.

2.2.2 Outlet Pipeline North of Fraser Gate Station

The alignment of the 762 mm (30 in., NPS 30) outlet pipeline segment between the Fraser gate station and South East Marine Drive at Elliott Street, which is also included in the scope of work for the present assessment, is shown in Figures 1 and 3. The ground surface slopes gently up from east to west along the pipeline segment between the Fraser gate station and the point where it crosses the railway line. In the vicinity of the pipeline-railway crossing, the ground surface elevation of Kent Avenue North is about 2.5 m higher than that of the railway bed, and the soil north of the railway line is retained by a lock-block gravity retaining wall. The topography in the area of the pipeline segment along Kent Avenue North slopes gently towards the Elliott Street intersection where the pipeline turns approximately 90 degrees (northward) along Elliott Street. Between Kent Avenue South and South East Marine Drive, Elliott street slopes steeply southwards at a gradient in the order of 10 to 12%.

2.3 BC Gas Piping Configurations at the Site

Buried piping within the station boundaries was not included in the scope of the regional assessment performed in 1993. In the present assessment, seismic response of both the above-ground and buried piping within the gate station were evaluated for the postulated earthquake-induced ground deformations.

The Fraser gate station serves as a transition point between the transmission and distribution portions of the BC Gas supply system. Gas supply to the Fraser gate station is from two transmission pipelines with diameters of 508 and 610 mm (20 and 24 inches, NPS 20 and NPS 24 pipes) that cross the North Arm of the Fraser River from the River Road gate station to the south as shown schematically in Figure 2. The profiles showing the invert levels of these transmission pipelines, plotted based on survey data provided by BC Gas, are presented in Figures 4 and 5.

A detailed plot plan of the gate station site, shown in Figure 6, illustrates the numerous above-ground structures and equipment associated with the operation of the gate station, as well as the range of buried pipe configurations present at the site. Key portions of the station piping include the gas heater pads, regulator station and metering building which lie on the westerly portion of the site. Maximum operating pressure for gas entering the gate station is 4020 kPa for the NPS 20 and NPS 24 pipelines. Gas leaves the metering station via a 30-inch pipeline (NPS 30) with a maximum operating distribution pressure of 1200 kPa.

The outlet line exits the north side of the station and turns west approximately 6 m north of the station fence line to parallel Kent Avenue South.

Buried portions of the piping within the gate station typically have 1 m of cover. Soil cover for portions of the pipeline between the gate station and Elliott Avenue vary from 1 to 3 m with a typical cover depth of 1 m. The greatest cover depths along this portion of the NPS 30 pipeline are approximately 45 m west of the station where the pipeline passes beneath a large concrete conduit and just north of the railway crossing.

2.4 Site Seismicity

The site is located in Seismic Zone 4 which is one of the zones of highest seismic risk as defined in the National Building Code of Canada (NBCC, 1990). The seismicity results from the thrusting of the offshore Juan de Fuca Plate beneath the continental North America Plate. There are three basic sources of earthquakes:

- Relatively shallow crustal earthquakes (depths in the order of 20 km);

- Deeper earthquakes (about 60 km depth) within the subducted plate; and
- Very large inter-plate earthquakes, often referred to as "mega-thrust" or "subduction" earthquakes.

Earthquakes within the first two categories (intra-plate) have been recorded in the region at regular intervals during the last several decades. The largest are those near Campbell River in 1946 (M = 7.3), near Olympia in 1949 (M = 7.1) and near Seattle/Tacoma in 1965 (M = 6.5). A very large earthquake is also reported to have occurred in central Washington state in 1872. Earthquakes from these sources are commonly included in probabilistic and deterministic seismicity models, such as the NBCC model.

Large subduction earthquakes have not occurred in the region in historic time. However, there is geological evidence that they have occurred in the past (possibly at 300 to 600 year intervals), and the measured accumulation of strain between the tectonic plates suggests that they should be expected in the future. The general consensus is that the magnitude of a large subduction earthquake would be in the order of 8.0 to 8.5; however, because of the greater epicentral distance from the Lower Mainland, the intensity of ground shaking is not expected to be greater than for the smaller intra-plate earthquakes. The primary concern with respect to the subduction earthquake is the duration of shaking, expected to be in the order of 2 to 3 minutes, or more than five times that of the intra-plate earthquakes.

2.5 Earthquake-Induced Hazards

Earthquake-induced hazards, in general, include ground motions and deformation, liquefaction, landslides, seiches and landslide-generated waves, and other related hazards such as flooding due to dyke failure and/or over-topping. Buried lifeline systems, such as the BC Gas pipeline network, are especially prone to damage during earthquakes resulting from ground failures and deformations associated with these events.

Since the site of BC Gas Fraser gate station is located in an area known to be underlain by loose/soft soils having a relatively high potential for liquefaction under earthquake loading, liquefaction-induced ground movements are considered to be the primary hazard to the pipeline and gate station. Based on the 1993 regional study, displacements of 2 m or more were predicted to occur at the Fraser gate station, even at moderate risk levels corresponding to a 1 in 475 year return period. Larger displacements, and incipient flowslide conditions, were predicted for the lower, 1 in 1,000 year and 1 in 2,000 year, risk levels.

In addition, liquefaction-induced ground subsidence, loss of bearing capacity, flotation and soil uplift were also identified as potential secondary hazards which could lead to pipeline failure at the site.

3.0 SEISMIC HAZARD PARAMETERS FOR SITE-SPECIFIC VULNERABILITY ANALYSIS

3.1 Risk Levels for Analyses

Considering the large costs associated with service restoration and the loss of revenue from consumers during system shutdown periods, we understand that only a very small risk of major service interruption under seismic loading is deemed acceptable to BC Gas.

Based on our discussions with BC Gas, it was agreed that for the present site-specific vulnerability assessment, risk levels corresponding to 100, 475, and 2,000 year return periods would be considered in evaluating the potential for damage.

3.2 Seismic Ground Motions

As a part of the 1993 regional study, the site-specific seismic hazard predictions were carried out using the BC Hydro seismicity model for the River Road gate station site which is located about 350 m south of the Fraser gate station on the south bank of the North Arm of the Fraser River. The BC Hydro assessment covered a wide range of risk levels as well as the variability inherent in the ground motion attenuation relation. Considering the close proximity, the seismic hazard predictions for the River Road site were considered applicable for the Fraser gate station site.

3.3 Predicted Ground Motions

The "best estimate" seismic hazard levels predicted for the site (based on the predictions for River Road gate station) using the BC Hydro model are summarized in Table 1 below.

Table 1

Firm Ground Motions for Fraser Gate Station

Return Period (yrs)	100	475	2000
Probability of Exceedance in 50 Years	40%	10%	2.5%
Peak Horizontal Firm Ground Acceleration (g)	0.09	0.20	0.34

4.0 GEOTECHNICAL INVESTIGATION

The geotechnical field investigation included both on-shore as well as off-shore drilling carried out in several phases. On February 23, 1995, six (6) sampled auger holes, AH95-1 and AH95-6, and four (4) electronic cone penetration tests (CPTs), CPT95-1 through CPT95-3 and SCPT95-4, were carried out at selected locations within the study area at the locations as shown in Figures 2 and 3. At the request of BC Gas, an additional electronic cone penetration test hole and an auger hole (CPT95-5 and AH95-7) were also put down within the north east quadrant of the gate station site (location shown in Figure 2) on March 24, 1995, to obtain soil information useful for planning of future development of the site, as well as to supplement the available geotechnical data for the present analyses. All the above testing work was performed using a truck-mounted drill rig.

Based on a preliminary review of the data from the above test holes, it was decided that additional geotechnical information on the soil conditions within the area south of the river bank (i.e., within the river) was required. Three over-water electronic cone penetration tests (CPT95-6 through CPT95-8) were carried out at the locations south of the river bank using a drill rig operated from a spudded barge. Standard Penetration Tests (SPTs) and a dynamic cone penetration test (DCPT), within a hollow-stem auger casing, were also carried out at the location of CPT95-8 to probe the soil conditions at depth, since CPT95 -7 and CPT95-8 had to be terminated at shallow depths due to refusal to penetration.

Since the observed soil conditions at the test hole AH95-3, put down at the intersection of Elliott Street and Kent Avenue North, were thought to be non-representative for the area (see Section 4.2 for details), an additional mud-rotary test hole BH95-1 was put down somewhat south-east of AH95-3 and outside the asphalt paved road area as shown schematically in Figure 3. The test hole was put down on June 14, 1995 using a track-mounted rig with a lower drill mast to maintain adequate clearance from a nearby overhead high-voltage power line.

4.1 Cone Penetration Test Holes

All electronic cone penetration tests (i.e., CPT95-1 through CPT95-8 and SCPT95-4) were put down to effective refusal to further penetration. The on-shore test holes that were located within the Fraser gate station compound were put down to depths ranging from about 6.5 to 13.5 m below the existing ground surface. During cone penetration testing, tip bearing, sleeve friction and pore pressure measurements were recorded at 0.05 m intervals of depth. Down-hole shear wave velocity measurements were also carried out at the test hole location SCPT95-4, and this data was used to compute the shear moduli of soil for ground motion response analyses.

The first offshore test hole CPT95-6 was put down to a depth of about 9.2 m below the mudline. However, test hole CPT95-7 which was put down about 25 m south of the CPT95-6, had to be terminated at a relatively shallow depth of about 3.1 m depth below the mudline. Since the depth to dense strata inferred from the other test holes was expected to be greater than the depth of refusal at CPT95-7, another test hole was attempted at the location CPT95-8 shown in Figure 2. Again, the testing had to be terminated at a depth of about 3.8 m from the mudline due to refusal. Obstruction to penetration due to a possible gravel layer was suspected; therefore, upon withdrawal of the CPT probe, the soils at this location were penetrated down to a depth of about 4.5 m below the mudline using a hollow-stem auger. After lowering a split spoon sampler inside the hollow-stem auger, a Standard Penetration Test was then carried out to sample the soil below the auger tip, and also to determine its relative density. A resistance to penetration of 13 blows were noted for the last 0.3 m penetration of sampler. The soil recovered indicated the presence of a wet, grey, fine to medium sand with some gravel at the sampled depth. Since the primary objective was to determine the depth to dense strata at this location, it was decided to further probe the soil by carrying out a dynamic cone penetration test (DCPT). Initially, the hollow-stem auger was pulled up from its previous position to a depth of about 3.4 m below the mudline, and the inside of the auger casing was then cleaned out. A DCPT was then carried out through the casing from this depth until refusal. The refusal to penetration of the dynamic cone occurred at a depth of about 6 m below the mudline. Based on the blow counts recorded during the last 0.6 m of DCPT testing prior to refusal, the depth to dense strata at this location was inferred to be 6 m below the mudline.

4.2 Sampled Test Holes

The seven sampled augerholes (AH95-1 through AH95-7), carried out on land, were advanced to the dense to very dense strata. The test holes carried out within the gate station compound were put down almost at the same locations of some of the cone penetration tests. The depth of the augerholes ranged from 2.3 to 15.2 m.

Test hole AH95-3 was put down at the intersection of Elliott Street and Kent Avenue North. Based on the visual classification of the soil samples retrieved, and the noted increase in resistance to augering, the depth to dense till-like dense strata at this location was determined to be about 3.8 m below the ground surface. The overlying material was found to consist of sand fill. Since this is an area where several below-ground services and manholes exist, it was considered likely that this test hole was located within a backfilled zone which would not be representative of the general soil conditions in the area. Because the depth to dense strata and the relative density of the overlying material would critically influence the site performance under seismic loading, further exploration was considered essential. In view of the above, an additional mud-rotary test hole BH95-1 (see Figure 3) was put down in an area which was expected to be outside the area of possible excavations made in the past.

All the field work was carried out under the full-time supervision of Golder Associates' personnel, who logged the soil conditions encountered at the various test holes and obtained representative samples for detailed examination and testing. The soil samples were transported to Golder Associates' laboratory in Burnaby for detailed inspection and selected laboratory testing.

5.0 SUBSURFACE SOIL CONDITIONS

Detailed descriptions of the soil conditions encountered during the investigation are presented in the Record of Borehole sheets and the cone penetration logs in Appendix I along with the measured down-hole shear wave velocity profile at test hole SCPT95-4. The results of laboratory index tests carried out on selected samples of soil are included on the Record of Borehole sheets. The results of grain size analyses carried out on selected soil samples recovered from the auger holes are presented in Appendix II.

The inferred soil conditions for the Fraser gate station and for the area along the alignment of the 760 mm discharge pipeline between the gate station and Elliott Street is discussed below.

5.1 Fraser Gate Station

A profile illustrating the inferred soil stratigraphy at the gate station site is shown in Figure 7.

The test hole data indicate that the upper soils within the station compound consist of about 1.7 to 2.7 m of loose to compact sand to sandy silt fill material. The test holes carried out within the northern part of the gate station compound indicate that the soils underlying the upper fill materials primarily consist of a layer of very soft to soft silt extending to depths in the order of 6 to 8 m below the ground surface. This silt layer is underlain by a compact to dense sand stratum, which in turn was found to overlie a very dense sand and gravel stratum at a depth of about 8.8 m below the ground surface at AH95-7.

As shown in Figure 7, the upper fills within almost the southern half of the site (the area approximately south of the existing regulator house) are underlain by loose to compact sand. At the test hole locations AH95-5 and AH95-6, which were located close to the southern fence of the station compound, this loose to compact sand layer extends to a depth of about 10.3 and 11.0 m below ground surface, respectively. Underlying these soils, compact to dense sand with a trace to some gravel was encountered. Test hole AH95-6 indicates that these strata are underlain by dense glacial till-like material which was encountered at a depth of about 14.0 m below the ground surface.

Test hole CPT95-6 carried out south of the river bank (within the river) also indicates the presence of sandy soils, below a 2 m thickness of silt and clayey silt, and extending down to a depth of about 9 m below the river bed. These materials were found to be underlain by a compact to dense soil stratum at this test hole location. Although the offshore test holes CPT95-7 and CPT95-8 had to be terminated at a relatively shallow depth due to inferred obstructions, the depth to dense strata estimated from the DCPT at CPT95-8 is consistent with that inferred through extrapolation of data from CPT95-6 and the other on-land test holes within the gate station compound.

The soil profile presented in Figure 7 represents the stratigraphy in the approximate north-south direction which is approximately parallel to the pipelines crossing the river. Based on our understanding of the geology of the area, we do not expect the soil profile to vary significantly in the east-west direction in the vicinity of the site.

The groundwater level within the gate station compound was noted to be at depths of about 1.0 m to 3.0 m below the ground surface based on our measurements during the period of the geotechnical investigation. The groundwater level at the site is expected to vary with the tidal variations of the adjacent North Arm of the Fraser River and seasonal precipitation and drainage conditions.

5.2 Outlet Pipeline North of Fraser Gate Station

Dense till-like soils were generally encountered at shallow depths (less than 1.5 m) at test holes put down within this pipeline segment. At augerhole AH95-1, put down within the right-of-way of Elliott Street, located almost half-way between South East Marine Drive and Kent Avenue South, a layer of compact to dense fine sand about 0.4 thick underlying sandy gravel fill was encountered at a depth of about 0.3 m below the ground surface. This sand layer was found to be underlain by a dense fine to medium sandy glacial till-like soil stratum at 0.76 m depth.

Similarly, a dense glacial till-like soil stratum was encountered at the location of augerhole AH95-2 underlying a 1.4 m thick surficial mineral fill layer. At test hole AH95-3, which was put down within the roadway at the intersection of Elliott Street and Kent Avenue North, a dense till-like dense stratum was encountered at a depth of about 3.8 m below the ground surface. The overlying material was found to be sand fill. The supplementary mud-rotary borehole BH95-1 put down south of the road pavement indicates that a glacial till-like stratum exists at a depth of about 1.5 m below the ground surface. The results from borehole BH95-1 indicate that the observed 3.8 m depth to the dense stratum at test hole AH95-1 is very likely a local anomalous depression of the till-surface, most probably caused by previous excavations made for utility installations as discussed in Section 4.2.

Except for moist soil conditions, no groundwater was encountered in the open test holes AH95-1 and AH95-3 based on our observations at the time of drilling. On the other hand, groundwater was encountered at a relatively shallow depth of about 0.3 m from the ground surface at the test hole AH95-2. Since a major portion of the pipeline alignment is located at the foot of the elevated area to the north, the groundwater level is expected to be encountered at shallow depths from the ground surface. The groundwater table will also vary with seasonal precipitation and drainage conditions.

6.0 GEOTECHNICAL ASSESSMENT OF SITE PERFORMANCE UNDER SEISMIC LOADING CONDITIONS - FRASER GATE STATION

The details of our geotechnical assessment of the seismic site performance of the area within the Fraser gate station and the river bank to the south are presented below.

6.1 Methodology

In order to provide the required geotechnical engineering input for the evaluation of pipeline vulnerability, analyses were carried out to assess the performance of the site soils under seismic loading conditions, including:

- Seismic ground motion response analyses to estimate the induced cyclic shear stress levels under various design loading conditions.
- Assessment of the liquefaction potential of the subsurface soils.

- Assessment of the post-seismic stability of the site.
- Evaluation of the earthquake-induced permanent lateral ground displacements.

The following subsections describe the above components of the analyses and the results obtained in detail.

6.2 Ground Motion Response Analyses

The dynamic ground motion response analyses were carried out using the one-dimensional wave propagation program SHAKE developed by H.B. Seed and his co-workers at the University California, Berkeley, U.S.A. (Schnabel, 1972). In SHAKE analysis, the nonlinear and hysteretic stress-strain behavior of the soil is modeled as equivalent linear visco-elastic using strain-dependent moduli and damping. Equivalence is achieved by an iterative procedure such that the moduli and damping values used are compatible with the computed strains.

The SHAKE analyses were carried out to compute the variation of equivalent cyclic stress ratios with depth (i.e., 0.65 times the maximum stress ratio) for use in the assessment of the liquefaction potential corresponding to seismic risk levels of 1:100, 1:475, and 1:2,000 year return periods.

Details on the selection of the other input parameters required for the SHAKE analysis are described below.

6.2.1 Earthquake records

The following firm ground motions recorded during the 1971 San Fernando earthquake (M6.4) were selected to represent the design earthquake shaking in the SHAKE analyses:

- Caltech - S90W component
- Lake Hughes Array #4 - S69E component
- Griffith Park Array - S00W component

In the SHAKE analyses, each earthquake record was scaled linearly with acceleration to represent the predicted base peak horizontal firm ground acceleration (PGA) for each risk level.

The acceleration-time histories of these earthquake records are shown in Figure 8. Response spectra (5% Damping) for these three earthquake input ground motions, derived by linear scaling of strong motion record to match the 1 in 2,000 year firm ground peak acceleration level, are presented in Figure 9. These records have been used in many ground motion response analysis studies in the Lower Mainland, including the 1991 Richmond Earthquake Task Force study (Task Force Report, 1991). In addition, based on our experience from SHAKE analyses carried out at several other sites in the Fraser Delta, these ground motions generally have been found to result in conservative estimates of induced critical stress ratios and surface ground motion predictions when compared to those computed from the earthquake time histories fitted to match the Uniform Hazard Response Spectra (UHRS) derived on a site-specific basis.

6.2.2 Soil Parameters

6.2.2.1 Maximum Shear Modulus

The maximum shear modulus (G_{max}) values in the SHAKE analyses were obtained from the measured shear wave velocities in the seismic cone penetration test (SCPT95-4). The following relationship was used to estimate the maximum shear modulus:

$$G_{max} = \rho \, V_s^2$$

where ρ is the mass density of the soil and V_s is the shear wave velocity.

The variation of shear wave velocity (V_s) with depth and the groundwater conditions assumed in the SHAKE column are presented in Figure 10.

6.2.2.2 Modulus Reduction and Damping Curves

The modulus reduction and damping curves used in the SHAKE analyses are shown in Figure 11. These curves are based on published data by Idriss (1990).

6.2.3 Results of Ground Motion Response Analyses

The variation of the computed equivalent cyclic stress ratio (CSR) and peak acceleration with depth for the 100, 475, and 2,000 year return periods are shown in Figures 12 through 14, respectively. The three curves plotted on each figure correspond to the SHAKE results using the three different ground motions. The equivalent CSR at a particular depth is defined as 0.65 times the peak cyclic shear stress divided by the vertical effective stress at that depth. Figure 15 shows the firm ground input acceleration and the peak accelerations at the ground surface predicted from the analyses. The results are also summarized in Table 2. The predicted peak accelerations at the ground surface for the 1:475 and 1:2,000 year return period levels show much higher acceleration levels than the median relationship proposed by Idriss (1990).

TABLE 2

Summary of Ground Motions Computed Using SHAKE

Risk Level (Return Period)	Input firm ground PGA (g)	Range of computed ground surface PGA from SHAKE (g)
1: 100 year	0.09	0.17 to 0.26
1: 475 year	0.20	0.37 to 0.39
1: 2,000 year	0.34	0.53 to 0.60

6.3 Assessment of Liquefaction Potential

The liquefaction potential of the subsurface soils at the site was assessed based on Seed et al. (1984) liquefaction resistance charts.

Based on a review of the distribution of magnitude contributions developed by BC Hydro for each seismic risk level, an earthquake magnitude of M7 was considered suitable for use in the liquefaction assessment irrespective of the seismic risk level. The use of a magnitude M7 for liquefaction assessment is also consistent with the current local design practice.

Since three ground motion time histories were used in the SHAKE analyses, three equivalent cyclic stress ratio vs. depth profiles were derived for each risk level. The equivalent cyclic stress ratio vs. depth profiles used in the liquefaction assessment, were derived by averaging the profiles obtained from the three earthquake time histories.

The SPT $(N_1)_{60}$ values required to prevent liquefaction for a given average equivalent cyclic stress ratio were then estimated from the Seed's charts for varying fines content (passing USS#200 sieve size). These $(N_1)_{60}$ values were converted to required cone bearing (q_c) values using the relationship by Robertson and Campanella (1986) with the overburden correction by Liao and Whitman (1985).

As per the Seed methodology, the required cone bearing values were then compared with the measured cone bearing values to delineate zones of potential liquefaction for soil containing less than 35% fines. The results are presented in Figures 16 through 18. It should be noted that the required q_c values shown in these figures are those having a factor of safety of 1 against liquefaction.

In fine-grained soils, such as the silty soil zone within the northern potion of the site (see Figure 7), the required cone bearing resistance to prevent liquefaction can be decreased below that required for clean sand. This is supported by previous laboratory as well as field observations indicating that the liquefaction susceptibility of soils decreases with increasing plasticity. In the absence of site-specific cyclic shear test data, the present practice is to use the Chinese criteria (Marcuson et al., 1990) for the assessment of liquefaction susceptibility of fine-grained soils. The Chinese Criteria are summarized in Table 4 below.

Table 3

Chinese Criteria for Assessment of Liquefaction Susceptibility of Fine-grained Soils

Soil Parameter	Condition for Liquefaction Susceptibility
Liquid Limit	< 35%
Water Content	> 0.9 * Liquid Limit
Fines Content < 0.005 mm Particle Size	<15%

The results of laboratory index tests carried out on representative samples of the upper sandy silt/silty sand layer are summarized below:

Sample	Liquid Limit	Water Content	% Fines < 0.005 mm
AH95-4: Sample 4	38.1%	42.7%	19%
AH95-4: Sample 5	-	-	19%
AH95-7: Sample 4	37.0%	39.6%	19%

Because of the relatively high (close to 20%) fines content below the 0.005 mm particle size (consistently observed in three samples) and the liquid limit values greater than 35%, the risk of liquefaction of the silty zone located within the northern zones of the site can be classified as low according to the Chinese criteria. This conclusion is also in agreement with our knowledge of previous data from cyclic undrained shear tests carried out on samples of similar geological origin, fines content, and liquid limits. Although the risk of liquefaction of the silty zone can be considered as low, some degradation of deformation moduli is expected due to cyclic shear strains and pore pressure generation under earthquake loading.

6.3.1 Subduction Event

The above analyses included only the impact of intra-plate crustal earthquakes (incorporated in the BC Hydro probabilistic model) and not the effect of a possible subduction earthquake on the Cascadia subduction zone off-shore of Vancouver Island. SHAKE analyses carried out considering the subduction earthquake as a separate earthquake scenario for the seismic vulnerability study of River Road gate station located 350 m south of the site, as well as for the seismic review of the BC Gas LNG plant, have indicated that that the critical stress ratio values and the ground surface accelerations estimated for the subduction event are lower than those estimated for a site-specific seismic hazard corresponding to the 1/2,000 annual risk level. Similar results were noted during our 1993/94 regional vulnerability assessment in the comparison of the ground displacements computed using the MLR method (which includes subduction events) for the 1:2,000 year return period level loading with those for the subduction scenario.

Considering that the depth of loose potentially liquefiable sandy soils at this site is generally limited to about the upper 12 m (cone penetration tip resistance of sandy and glacial till-like soils $Q_c > 150$ bars generally below these depths), and that the zone of liquefaction estimated for the 1:2,000 year return period level loading extends to such depths in the sandy soil zones, we are of the opinion that the zone of potential liquefaction and the associated ground movement hazard for the subduction event is adequately encompassed by the assessment for the 1:2,000 year return period level earthquake. As such, additional SHAKE analyses to investigate the subduction event as a separate scenario were not considered to be required.

6.4 Post-Liquefaction Slope Stability

Having identified the zones of potential liquefaction, the post-liquefaction stability of the gate station compound was analyzed using the computer code XSTABL. A cross-section developed in the approximate north-south direction (i.e., approximately perpendicular to the crest of river bank) was considered in the slope stability analyses.

Cross-sections showing the soil layering and the slip surfaces considered in the analyses are presented in Figures 19 through 24. Both circular and non-circular failure surfaces were analyzed to investigate the potential for a flow slide condition at the site. A factor of safety ≤ 1.0 under post-liquefaction soil conditions, without application of seismic inertia forces, would indicate a high risk of a flow slide as a result of earthquake shaking.

The parameters used in the stability analyses are summarized in Table 4.

Table 4

Soil Parameters Used in Slope Stability Analyses

Soil Layer	Density (kN/cu.m.)	Cohesion (kPa)	Friction Angle (deg)
Upper FILL Materials	19.0	0.0	32.0
Loose to compact Liquefiable SAND	19.0	0.0	11.3
Very soft to soft SILT	18.0	0.0	25.0
Compact to dense SAND	19.0	0.0	35.0
Dense to very dense glacial TILL	22.0	0.0	39.0

The input parameters for the non-liquefiable materials were derived based on the results of test hole data obtained during the geotechnical investigation and previously established correlation in the literature. The shear strength parameters for potentially liquefiable zones were selected based on recently reported data from laboratory post-cyclic monotonic simple shear tests carried out by BC Hydro on frozen samples of sand recovered from the Duncan Dam site (Pillai and Stewart, 1994) and those reported by the University of British Columbia on reconstituted samples of Fraser River sand (Sivathayalan, 1991). These results indicate that the use of an S_r/σ'_v of 0.2 (effective friction angle of 11.3 degrees) is reasonable for soils having relative densities ranging from 40% to 60% (i.e., for clean sands with $(N_1)_{60}$ ranging from about 8 to 18 based on Skempton (1986)).

The results of the analyses indicate that, for the failure surfaces considered, the circular surface as shown in Figure 19 is the most critical in terms of slope stability. As noted in the figure, the computed post-liquefaction factor of safety without application of any seismic inertia force was computed to be 0.91. Since this computed factor of safety is below unity, it can be concluded that a flow slide condition leading to "unlimited" deformations at the site is likely. Although the zone of potential liquefaction corresponding to 1 in 100 year return period level seismic loading was slightly smaller than those for the 1 in 475 and 1 in 2,000 year return period levels, the outcome of the analyses in terms of critical slip surface was essentially unchanged since the critical slip surface passed through a zone that is potentially liquefiable under all three risk levels considered in the analyses.

Since the post-seismic factor of safety (without inertia forces applied) is below unity, the above results also indicate a yield acceleration of zero (0.0 g) for the critical failure surface. (Note: The yield acceleration is the minimum horizontal earthquake acceleration coefficient when applied at the center of gravity of the sliding mass which would reduce the factor of safety against sliding to unity). Generally, the larger the difference between the predicted PGA for the site and the critical yield acceleration, the greater are the expected ground movements. A yield acceleration of zero (i.e., post-seismic static FOS $\leq$ 1.0) indicates a high risk of a flow slide as a result of earthquake shaking.

6.5 Liquefaction-Induced Lateral Ground Displacements

Although a flow-slide leading to large ground deformations was identified for the southern part of the site (within 30 m from the crest of river bank) for earthquake loading corresponding to all three risk levels, limited ground displacement analyses were carried out to obtain a better understanding of the magnitude and patterns of the relative ground movements in the northern parts of the site, as input for the structural vulnerability assessment of the facilities. The liquefaction-induced free-field ground displacements were calculated using the following methods:

a) Computer program DISPLMT developed by Houston et al. (1987) using the Newmark (1965) sliding block method; and

b) Empirical MLR method developed by Bartlett and Youd (1992)

6.5.1 DISPLMT Analysis

The free-field ground displacements corresponding to the slip circles identified in Figures 19 through 24 were estimated using the computer program DISPLMT.

In order to compute the displacements, the program requires the "static" factor of safety and the yield acceleration of the slope (using post-liquefaction soil parameters) and an estimate of the acceleration-time history of the failure zone with respect to the soil mass below. These were obtained from the slope stability analyses discussed in the previous Section 6.4. The acceleration-time histories to be used in the DISPLMT program were computed using the one-dimensional SHAKE analysis according to the guidelines given by Houston et al. (1987).

The predictions from the DISPLMT analyses for the three different risk levels are presented in Figure 25. As shown, for the seismic loadings corresponding to all the risk levels considered in the study, large ground displacements in excess of 3 m which would lead to a flow slide towards the river are predicted to influence an area extending to about 30 m north from the crest of the river bank. For the seismic risk levels of 1 in 475 and 1 in 2,000 year return periods, lesser, but still significant, ground displacements are predicted to influence an area extending to about 40 m from the crest of the river bank.

The DISPLMT program assumes that the failing soil mass as well as the underlying soils will act as rigid bodies under seismic loading and, therefore, does not account for the flexibility of the soil. However, the computed deformations from the program give a general indication of the expected ground movements, and confirm that there is a significant deformation hazard at the site.

6.5.2 MLR Method

The empirical MLR method developed by Bartlett and Youd (1992), which was used in the regional study, was also used to evaluate the liquefaction-induced ground movements at the site. This method has been developed based on Multiple Linear Regression (MLR) analyses of earthquake, topographical, soil type and geological data associated with liquefaction-induced lateral spreads resulting from eight major earthquakes. In the MLR model, two equations have been developed to predict median values of permanent lateral ground surface displacements at sites susceptible to liquefaction in the vicinity of river banks or free faces (Free-Face equation) and for general sloping conditions (Ground Slope Equation). The former equation was used in the predictions for the present assessment.

An earthquake magnitude of M7 was assumed in the predictions. Using the ground surface acceleration predictions for each risk level from SHAKE analyses, the corresponding equivalent source distance was back-calculated using the chart proposed by Bartlett and Youd (1992).

The mean value of the permanent lateral ground surface movements computed using the MLR soft soil approach also resulted in very large ground movements (in excess of 5 m) within the southern area of the site, in general agreement with the results of the DISPLMT analyses. The computed ground displacements for the silty zones within the approximate northern half of the site were low (i.e., < 0.1 m), again, in agreement with the DISPLMT results.

In summary, the results of our analyses indicate that there is a significant risk of large ground movements, indicating possible flow slide conditions, within the southern portion of the regulator station, even under the 1 in 100 year return period loading.

6.6 Liquefaction-Induced Vertical Ground Displacements

Along with the lateral ground movements, significant vertical ground movements are also expected within the southern area of the site due to translation of the soil mass. These displacements are generally expected to reduce with increasing northward distance from the crest of the river bank.

The vertical ground movements due to translation of the soil mass are expected to be reduced significantly, if ground improvement as discussed in Section 10, is undertaken to mitigate the liquefaction induced-lateral movements. However, the ground surface displacements due to dissipation of pore pressures (i.e., consolidation of the liquefied soils) should still be expected within the potentially liquefiable areas north of the area of ground improvement. Saturated liquefiable soils are expected to generate excess pore pressures under seismic loading and settle as the pore pressures dissipate. Tokimatsu and Seed (1987) proposed a method of estimating settlement of loose sands under earthquake loading. The applicability of this method had been illustrated by comparing the predicted displacements with those observed from field measurements.

Using the Tokimatsu-Seed (1987) empirical charts, additional downward ground movements up to 0.2 m are estimated due to consolidation of liquefied soils within the potentially liquefiable areas north of the area of ground improvement.

7.0 GEOTECHNICAL ASSESSMENT OF SITE PERFORMANCE UNDER SEISMIC LOADING CONDITIONS - OUTLET PIPELINE NORTH OF FRASER GATE STATION

The results of the geotechnical investigation indicate the presence of dense to very dense (glacial till-like) soil strata, underlying granular fills, at shallow depths along the pipeline alignment between South East Marine Drive and the pipe elbow at the south side of the railway crossing (i.e., near test hole AH95-2, see Figure 3). The depth to dense strata appears to generally increase from a depth of about 1.4 m at the railway crossing to a depth of about 6 m at the Fraser gate station to the east. Based on the test hole data, and also our understanding of the geology of the area, we infer that the soils above the dense strata primarily consist of fills in the order of 1 to 2 m in thickness underlain by soft silty soils similar to those encountered at test holes AH95-4/CPT95-1, CPT95-2, and AH95-7/CPT95-5.

Based on our evaluation of the soil conditions for the area, we are of the opinion that the risk of liquefaction-induced hazards under seismic loading along the outlet pipeline is low. The earthquake-induced ground surface displacements along the segment of the outlet pipeline between the Fraser gate station and the railway crossing are not expected to be more than 0.3 m. Lesser ground movements are expected along Kent Avenue and Elliott Street north of the railway crossing.

8.0 SUBSOIL PARAMETERS FOR STRUCTURAL VULNERABILITY ANALYSIS OF PIPELINE SYSTEM

The soil parameters given in Table 5 are recommended for use in deriving input parameters for the soil-pipe interaction modeling in the structural assessment of pipeline vulnerability. The material parameters presented herein correspond to the upper three stratigraphic zones shown in Figure 7.

TABLE 5

Soil Parameters for Vulnerability Analyses

Soil Type	Cohesion (kPa)	Friction Angle (deg)	Density (kN/cu.m.)	Coefficient of earth pressure at rest (Ko)
Upper granular FILLS	0	30-32	19.0	0.5
Loose to Compact SAND				
(a) Pre-seismic	0	28-31	19.0	~0.5
(b) Post seismic	10-15	0	19.0	~1
Very Soft to Soft SILT				
(a) Pre-seismic	20-30	0	18.0	~0.5
(b) Post-seismic	15-25	0	18.0	~0.6

9.0 ASSESSMENT OF PIPING VULNERABILITY

Vulnerability of the piping within the scope of this study was estimated based upon the results of generic vulnerability analyses performed in support of the 1993 regional study, site-specific soil strength information reported in Table 5, experience with detailed evaluations of similar configurations, and observation of pipeline performance in past earthquakes where permanent ground deformation was present.

In the regional risk assessment performed in 1993, due to lack of site-specific soils data, soil parameters for piping vulnerability analyses were developed assuming two generic soil types most likely to be encountered within the upper 3 to 4 m of ground surface. The two soil types were defined as Material Types I and II. Material Type I was designated to represent loose sand or silty sand and silts, and Material Type II was to represent soft peat and silty soil conditions.

In the regional assessment the Fraser gate station location was treated as a Type I soil site. The ultimate axial and lateral soil loading used in the regional assessment are presented in

Table 6. Those derived based on the current site-specific soils data are also presented in the same table for comparison purposes.

The acceptable permanent ground deformation limits for Type I and Type II soils used in the regional risk assessment are summarized in Table 7. In general, ground deformation limits were rounded off to numbers that reflected the overall level of accuracy associated with the regional assessment.

TABLE 6

Comparison of Ultimate Soil Strengths for Vulnerability Assessment

1993 Regional Study	20-inch	24-inch	30-inch
Axial Soil Force, kN/m	25	30	37
Lateral Soil Force, kN/m (Type I Soil)	66	83	108
Lateral Soil Force, kN/m (Type II Soil)	137	168	216
Current Investigation			
Axial Force, kN/m	22	27	35
Lateral Force, kN/m	106	123	149

TABLE 7

Permanent Ground Deformation Limits - 1993 Vulnerability Assessment

Pipe Size	20 x .281		24 x .344		30 x .418	
Estimated Probability of Rupture	10%	50%	10%	50%	10%	50%
Lateral Offset on Straight Pipe, Type I Soil (m)	4	4	0.7	4	0.7	4
Lateral Offset on Straight Pipe, Type II Soil (m)	4	4	4	4	1.5	4
Length of Straight Pipe Subject to Longitudinal Lateral Spread Movement (m)	200	300	190	290	180	270
Offset at Elbow, Type I Soil (m)	0.1	0.5	0.2	0.5	0.1	0.5
Offset at Elbow, Type II Soil (m)	0.2	1	0.3	1	0.2	1

Another consideration in assessing the vulnerability of pipelines in the current investigation relates to the strength and ductility associated with the pipeline itself. In the regional assessment, all pipelines were assumed to be free from defects and have butt-welded connections capable of developing the ultimate strength of the pipe. This assumption is generally not valid for station piping where numerous bolted flange connections are necessary to accommodate connections to equipment such as valves, heaters, orifice meters, etc. Deformation limits associated with the 10% probability column are more appropriate for assessing the impact of ground deformations on station piping.

Based on the variation of ground deformation capacity with soil type and the soil strength information presented in Table 6, approximate ground deformation capacities for assessment of the Fraser gate station piping are listed in Table 8. These capacities are very approximate and serve primarily as a means to judge the relative severity of the site-specific estimates of ground deformation.

TABLE 8

Approximate Permanent Ground Deformation Capacities for Assessment of Piping Vulnerability - Current Investigation

Pipe Size	20-inch	24-inch	30-inch
Lateral Offset on Straight Pipe (m)	4	2	1
Length of Straight Pipe Subject to Longitudinal Lateral Spread Movement (m)	200	190	180
Offset at Elbow (m)	0.15	0.25	0.15

The results of the vulnerability assessment are presented with respect to the area within the fenced boundary of the gate station and the portion of the buried distribution pipeline between the gate station and the intersection of Marine Drive and Elliott Avenue.

9.1 Assessment of Piping Within the Gate Station Fenceline

Maximum computed surface ground deformation contours from the DISPLMT analyses for the 2000-year return period are superimposed with the gate station plot plan in Figure 26. It is clear from Figure 26 that differential displacements at the Fraser gate station from earthquake-induced liquefaction exceed the estimated capacity of the pipelines presented in Table 8 above by an order of magnitude. Given the large exceedance of pipeline capacity, the only remedial measures deemed practical for the existing site relate to improving the ground conditions. Provided that ground improvements can reduce the potential earthquake-induced permanent ground deformations to less than about 15 cm, no modification of the existing station piping is judged necessary.

Vertical deformations due to liquefaction of untreated areas are predicted to occur. While these are not expected to be of serious concern with respect to the transmission pipelines, consideration will have to be given to smaller diameter station piping and particularly connections to piled structures, during detailed design of the remedial treatment works.

9.2 Assessment of Piping Outside of the Gate Station Fenceline

Pipeline alignments outside of the Fraser gate station fenceline and included in the scope of the present assessment include the portion of the 762 mm (30 inch) intermediate pressure line between the gate station and the intersection of Elliott Avenue and Marine Drive and the portion of the two transmission pipelines crossing beneath the North Arm of the Fraser River.

9.2.1 Outlet Pipeline North of Fraser Gate Station

The maximum expected earthquake-induced permanent ground movement for the 762 mm pipeline is estimated to be 0.3 m. This is well below the estimated capacity of the pipeline for displacements occurring away from elbows and about twice the estimated capacity for station piping having displacements occurring near elbows. Two key conditions provide a basis for a higher allowable elbow deformation capacity.

The absence of bolted flange connections for this portion of the alignment justifies increasing the deformation capacity closer to that associated with higher bending strains. For this portion of the pipeline alignment, increasing the deformation capacity of the elbow to at least 0.4 m is judged reasonable.

While the maximum amount of deformation is estimated to be 0.3 m, it is the differential displacement that is of importance for the response of the pipeline in the vicinity of an elbow. The differential displacement between portions of the pipeline east and west of the railway crossing is expected to be less than 0.3 m.

For these reasons, no soil or pipeline improvements are judged necessary for the 762 mm intermediate pressure pipeline in this area.

9.2.2 Transmission Pipelines South of Fraser Gate Station

Given the magnitude of the maximum permanent ground deformations estimated for the gate station site, failure of the transmission pipelines beneath the river is possible. If ground improvements at the site are carried out, there is still a potential for large ground deformations south of the zone of ground improvement and within the north bank of the river crossing. The impact of such ground failure would be to place tensile loads on the transmission lines that would be resisted by their effective anchorage in the improved soil region of the gate station. Approximately 150 m of pipeline is impacted by such southward movements conservatively assuming that the deformations extend to the midpoint of the river crossing. Assuming symmetric pipeline response, the tensile force estimated to be transferred to the point of anchorage at the site is 1650 kN. For a 508 mm (20 inch) pipeline with a 7 mm (0.281 inch) wall, this represents a tensile stress of 144 kPa. This is on the order of 50% of the minimum tensile yield stress of the pipe and is judged not to pose a credible threat of rupture. The results are similar for the 610 mm (24 inch) pipeline.

The above qualitative assessment is believed to be conservative based on the following:

1. The analysis of permanent ground deformation indicates large ground movement (greater than 5 m) which is consistent with flow failures. The soil loading on the pipeline under such conditions will be much less than what has been assumed above.

2. Considering the soil movement to extend to the center of the river crossing is believed to represent an upperbound estimate.

3. The pipeline wall thickness is greater than 7 mm for the river crossing.

For these reasons, modification of the pipeline within the gate station to account for possible soil loading occurring at the river crossing is not expected to be necessary. This conclusion needs to be carefully reviewed as part of detailed design of soil remediation measures to confirm that the pipe within the station boundary is effectively anchored.

The transmission pipelines crossing the North Arm of the Fraser River are coated with 76 mm of concrete. This coating assures the pipe is negatively buoyant in water. Within the soil expected to liquefy, the pipelines will be subject to modest uplift forces as a result of relative buoyancy. However, the liquefied soil is considered to have sufficient residual shear strength to preclude relative pipe movement as a result of buoyancy.

It is possible that earthquake-induced ground movements could adversely affect pipeline soil cover. The expected large deformations near the river bank imply flow failures are likely. River currents would cause flowing soils to be transported downstream. Similar ground failure conditions would be expected to occur at other locations upstream of the gate station leading to sediment deposition as well as potential for debris to traverse the pipeline locations. The net effect on pipeline soil cover is unknown.

10.0 REMEDIAL TREATMENT MEASURES - FRASER GATE STATION

The pipeline vulnerability assessment carried out by EQE International (see Section 9.0) indicates that the existing piping configuration at the Fraser gate station will be vulnerable to damage even under the seismic loading conditions corresponding to the 1 in 100 year risk level.

As discussed in Section 9.0, a program of ground improvement appears to be the most suitable in terms of reducing the pipeline vulnerability at the gate station. Potential remedial treatment for the site is discussed below.

10.1 Conceptual Ground Improvement Measures

Various ground improvement techniques could be considered to reduce the risk of liquefaction and magnitude of ground deformations. Based on our geotechnical assessment, we are of the opinion that the densification of selected soil zones within the river bank area and/or within the gate station compound would be effective in minimizing the potential ground movements under earthquake loading. Such treatment of soils should preferably be carried out to form a non-liquefiable barrier (or barriers) aligned perpendicular to the predominant direction of potential ground movements.

10.1.1 Ground Improvement Requirements

The slope stability program XSTABL was used to investigate the effectiveness of ground improvement in reducing the expected liquefaction-induced ground displacements at the site. A design concept assuming the densification of a rectangular area parallel to the dyke alignment was investigated. The width of the densification zone was assumed to be 10 and 20 m (as shown in Figures 27 and 28), and the treatment was assumed to extend to the predicted full depth of liquefaction.

The results of the slope stability analyses are presented in Figures 27 and 28. These results illustrate that an increase in the yield acceleration, leading to a decrease in liquefaction-induced ground displacements, could be achieved by densification of selected areas.

The ground displacements corresponding to the above configurations of assumed ground improvement were again predicted using the computer program DISPLMT. The predictions were made only for the case of the critical slip surface. The results of the computed displacements are compared with those computed for "as-is" ground conditions in Table 9. The impact of ground improvement is shown by the significant reduction in the predicted ground deformations.

TABLE 9

Effect of Ground Improvement on Computed Earthquake-Induced Ground Displacements

	Computed Lateral Displacement within 20 m of River Bank (m)					
Earthquake Return Period	1:100 yr		1:475 yr		1:2,000 yr	
Level of Ground Improvement	Mean	Max.	Mean	Max.	Mean	Max.
No grnd. imprvmt. ("as-is" condition)	> 3 m	> 3 m	> 3 m	> 3 m	> 3 m	> 3 m
Ground imprvmt. Barrier Width = 10 m	< 0.1 m	< 0.1 m	< 0.1 m	0.2 m	0.3 m	0.5 m
Ground imprvmt. Barrier Width = 20 m	< 0.1 m	< 0.1 m	< 0.1 m	< 0.1 m	<0.1 m	0.1 m

These results indicate that the introduction of a densified barrier, likely in the order of 15 to 20 m wide, would significantly reduce the expected large earthquake-induced ground movements in the vicinity of the gate station.

As discussed in Section 6.5.1, the DISPLMT program assumes that the failing soil mass, as well as the underlying soils, will act as rigid bodies under seismic loading and, therefore, does not account for the flexibility of soil. Therefore, it is possible that the computed ground displacements from the DISPLMT method may be under-predicted. More rigorous computer analysis using the program SOILSTRESS (Byrne et al., 1992), which is capable of modeling the flexibility of soil, would be required to estimate ground movements and optimize treatment zones at the detailed design stage.

10.1.2 Techniques of Ground Improvement

Several methods are available for improving the liquefaction resistance of soils. These methods include:

- Vibro-replacement (installation of stone columns);
- Compaction Piles;
- Dynamic Compaction;
- Blast densification; and
- Compaction Grouting.

The selection of the most suitable ground improvement technique is governed by several factors, such as soil conditions, equipment space restrictions, pipeline protection issues, environmental regulatory requirements, land availability etc. Based on our geotechnical requirements, the method of vibro-replacement (involving the installation of "stone columns") is considered to be the most suitable technique of ground densification for use at the Fraser gate station site. It is recommended that the proposed treatment be reviewed to ensure that it will satisfy other considerations and constraints.

The vibro-replacement method essentially involves installation of stone columns using a vibratory probe. Typically the stone columns are installed in a triangular pattern with a center to center spacing ranging from 2 to 3 m. In addition to the densification achieved, the stone columns would assist in the dissipation of excess pore pressures generated due to earthquake loading by providing vertical drainage paths. This method has been found to be very effective in densifying sandy soils where densification by vibration is easily achieved.

The focus of the results and the discussions presented above has been to illustrate the potential site mitigation options on a conceptual basis. The actual sizing and configuration of ground improvement zones, and selection of the most suitable technique should be determined during detailed design, with due consideration given to the other factors identified above.

10.1.3 Preliminary Cost Estimates

In order to provide an order-of-magnitude estimate of the costs involved in ground improvement, we have carried out a preliminary cost analyses for an assumed densification zone extending over a 15 m x 100 m area parallel to the river bank (i.e., extending from the east end of the gate station compound to the west end). Based on the current rates quoted by specialist contractors, the basic cost of installation of vibro-columns would be in the order of $60 per linear meter of column installed. If the densification of a 15 m wide barrier described as above is considered, the plan area of ground improvement would be in the order of 1,500 sq.m., and considering the potentially liquefiable zone, the average depth of treatment would be about 10 m. Based on the above rates, and assuming a densification pattern of 2.5 m triangular center to center spacing, the approximate cost of densifying such an area using vibro-replacement, including mobilization, was estimated to be in the order of $200,000 in June 1995, at the time the evaluation of mitigation options was carried out. The above approximate cost has been estimated based on our previous experience on densification work carried on similar projects and unit rates available from specialist contractors.

In addition to the direct costs related to vibro-replacement, it is recommended that budget allowance also be made for costs involved in ancillary works that would be required in performing the ground improvement. Based on our understanding of the site, some of the ancillary works which we can identify at this time are listed below:

a) relocation of street lighting and fences, site clean-up;

b) removal of rip-rap and construction of a gravel platform for accessing the river bank;

c) possible regrading of shoreline and restoration of rip-rap during restoration of river bank to satisfy geotechnical, hydrotechnical, as well as environmental requirements;

d) other environmental controls (e.g., silt curtains) and mitigative measures (e.g., restoration of vegetation);

e) any gate station equipment relocation; and

f) engineering construction inspection and monitoring.

Detailed estimation of costs related to most of the ancillary work above is not practical without information on final design details and the knowledge of other operational and regulatory requirements. However, based on the limited information available at present and further to some discussions with specialty contractors, consideration should be given to inclusion of an additional budgetary allowance of $200,000 to $250,000 to cover these items.

11.0 CLOSURE

We trust that that this report provides sufficient information for your current requirements. Should you have any questions or require additional information please do not hesitate to contact us.

GOLDER ASSOCIATES LTD.

D. Wijewickreme, Ph. D., P. Eng.
Senior Geotechnical Engineer

T.P. Fitzell, P. Eng.
Principal

DW/TPF/vw
942-1211A/1070

J:\RPT-97\JAN\DW21211A.DOC

REFERENCES

Bartlett, S.F., and Youd, T.L. (1992). *Empirical Analysis of Horizontal Ground Displacement Generated by Liquefaction-Induced Lateral Spreads*, Technical Report NCEER-92-0021, National Center for Earthquake Engineering Research, State University of New York at Buffalo.

Byrne, P.M., Jitno, H., Salgado, F. (1992). *Earthquake Induced Displacements of Soil-Structures Systems*, Proceedings, Tenth World Conference in Earthquake Engineering, Madrid, Spain, pp. 1407-1412.

Houston, S. L., Houston, W. N. and Padilla J. M. (1987). *Microcomputer-Aided Evaluation of Earthquake Induced Permanent Slope Displacements*, Microcomputers in Civil Engineering 2, Elsevier Science Publishing Co., pp. 207-222.

Idriss, I. (1990). *Response of Soft Soil Sites During Earthquakes*, H. Bolton Seed Memorial Symposium, Vol. 2.

Liao, S.C. and Whitman, R.V. (1985). *Overburden Correction Factors for SPT in Sand*, J. Geotechnical Engineering Division, ASCE, Vol. 112, No. GT3.

Marcuson III, W.E., Hynes, M.E., and Franklin, A.G. (1990). *Evaluation and Use of Residual Shear Strength in Seismic Safety Analyses of Dams*, Earthquake Spectra, Vol. 6, No. 3, August.

NBCC - National Building Code of Canada (1990). National Research Council.

Newmark, N.M. (1965). *Effects of Earthquake on Dams and Embankments*, Rankine Lecture, Geotechnique 15(2), pp. 139-160.

Pillai, V.S. and Stewart, R.A. (1994). *Evaluation of Liquefaction Potential of Foundation Soils at Duncan Dam*, Canadian Geotechnical Journal, Vol. 31, No. 6, pp. 951-966.

Robertson, P.K. and Campanella, R.G. (1986). *Guidelines for Use, Interpretation and Application of the CPT and CPTU*, Soil Mech. Series 105, Dept. of Civil Engineering, University of B.C.

Schnabel, P.B., Lysmer, J. and Seed, H.B. (1972). "*SHAKE - A Computer Program for Earthquake Response Analysis of Horizontally Layered Sites*". Report No. EERC 72-12, Earthquake Engineering Research Center, University of California, Berkeley.

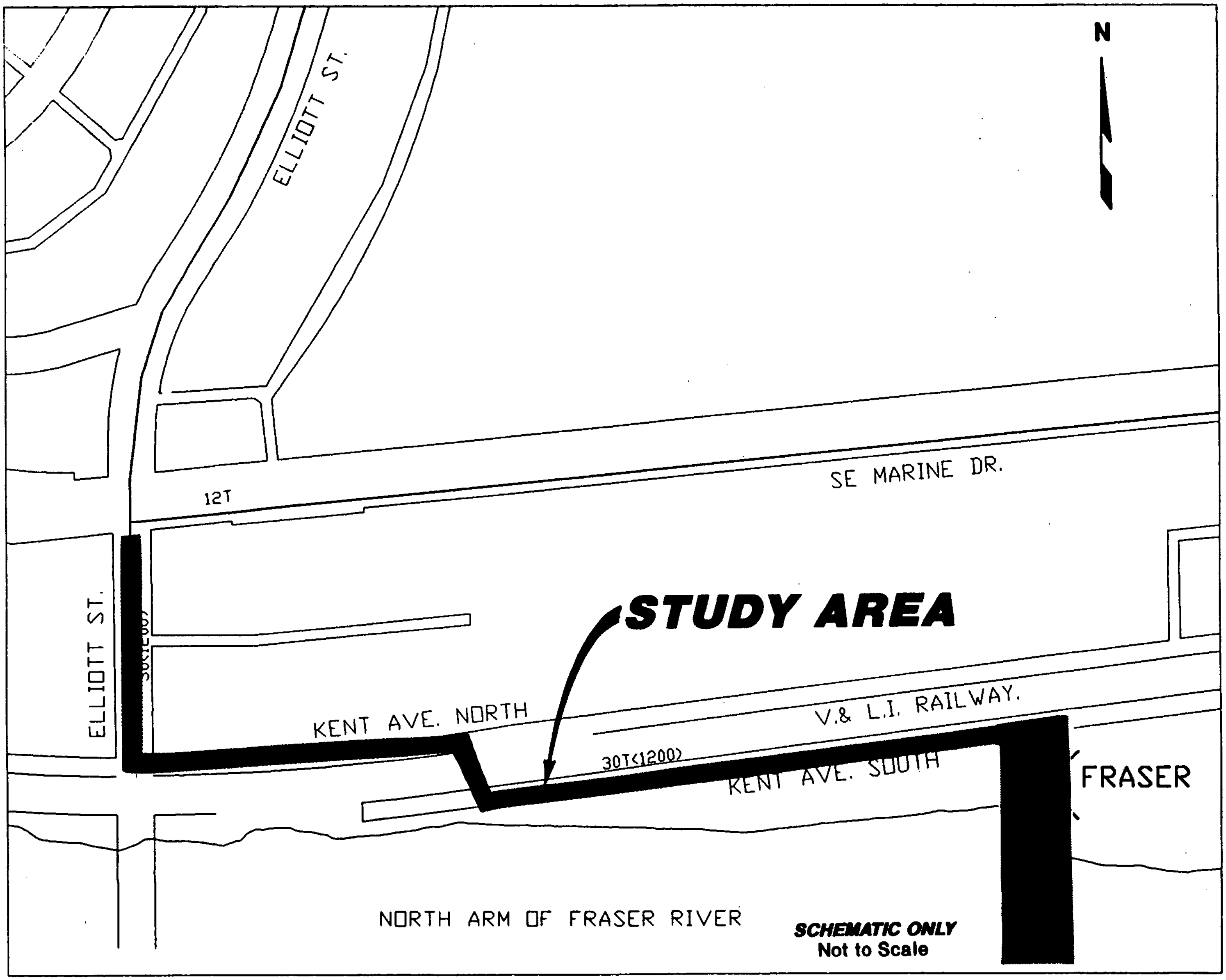
Seed, H.B., Tokimatsu, K., Harder, L.F. and Chung, R.M. (1984). *The Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations*, J. Geotech. Engineering Division, ASCE, Vol. 111, No. 12, pp. 1425-1445.

Sivathayalan, S. (1991). *Static, Cyclic, and Post Liquefaction Simple Shear Response of Sands*, M.A.Sc. Thesis, Department of Civil Engineering, University of British Columbia.

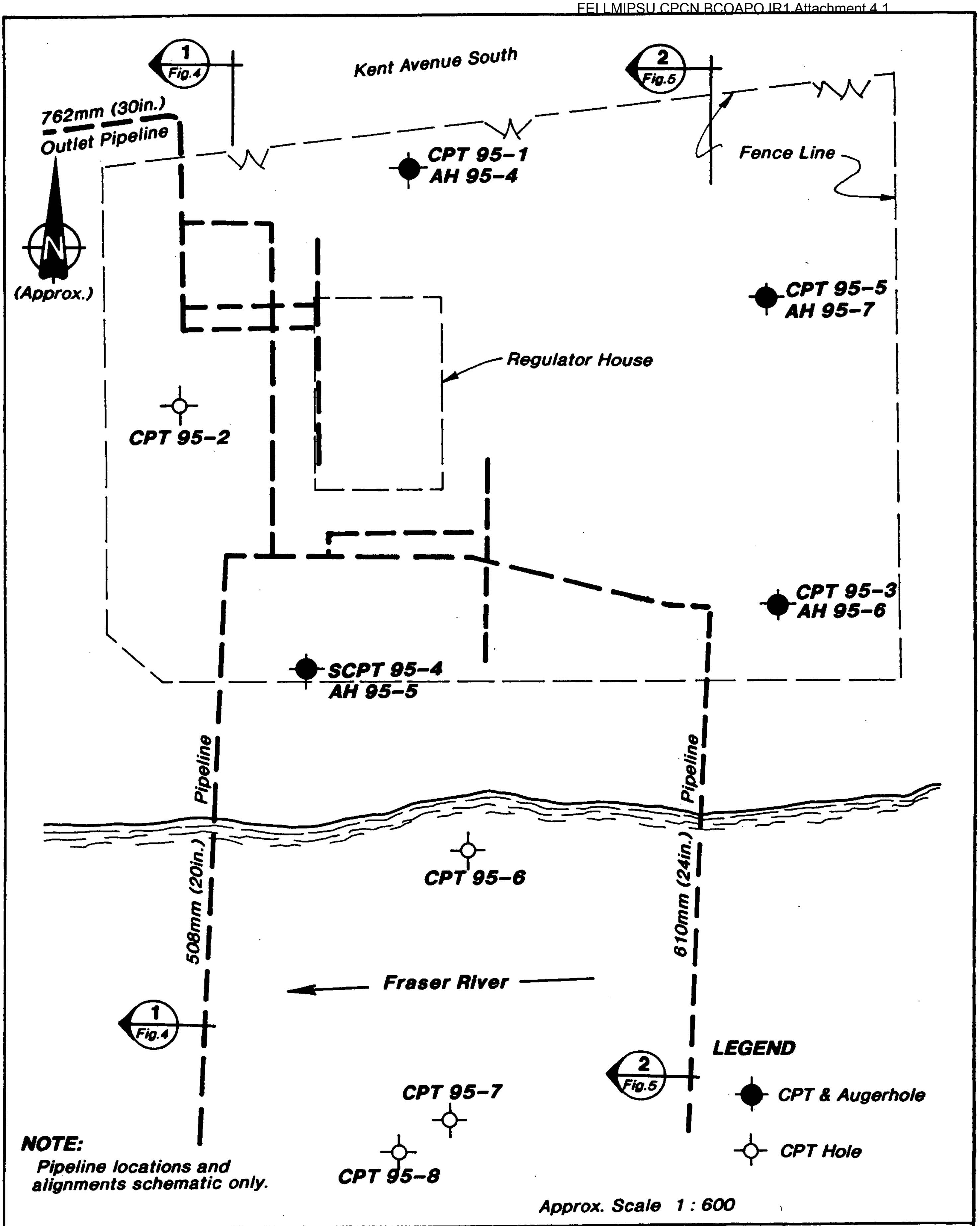
Skempton, A. W. (1986). *Standard Penetration Test Procedures and the Effects in Sands of Overburden Pressure, Relative Density, Particle Size, Aging, and Overconsolidation*, Geotechnique 36, No. 3, pp. 425-447.

Task Force Report (1991). *Earthquake Design in Fraser Delta.*

Tokimatsu, K., and Seed, H.B. (1987). *Evaluation of Settlements in Sands due to Earthquake Loading*, Journal of Geotechnical Engineering, ASCE, Vol. 113, No. 8, pp. 861-878.



Project No. 942-1211A **Drawn** EQE **Reviewed** D.W. **Date** July '95	Golder Associates	**KEY PLAN**	**Figure** 1



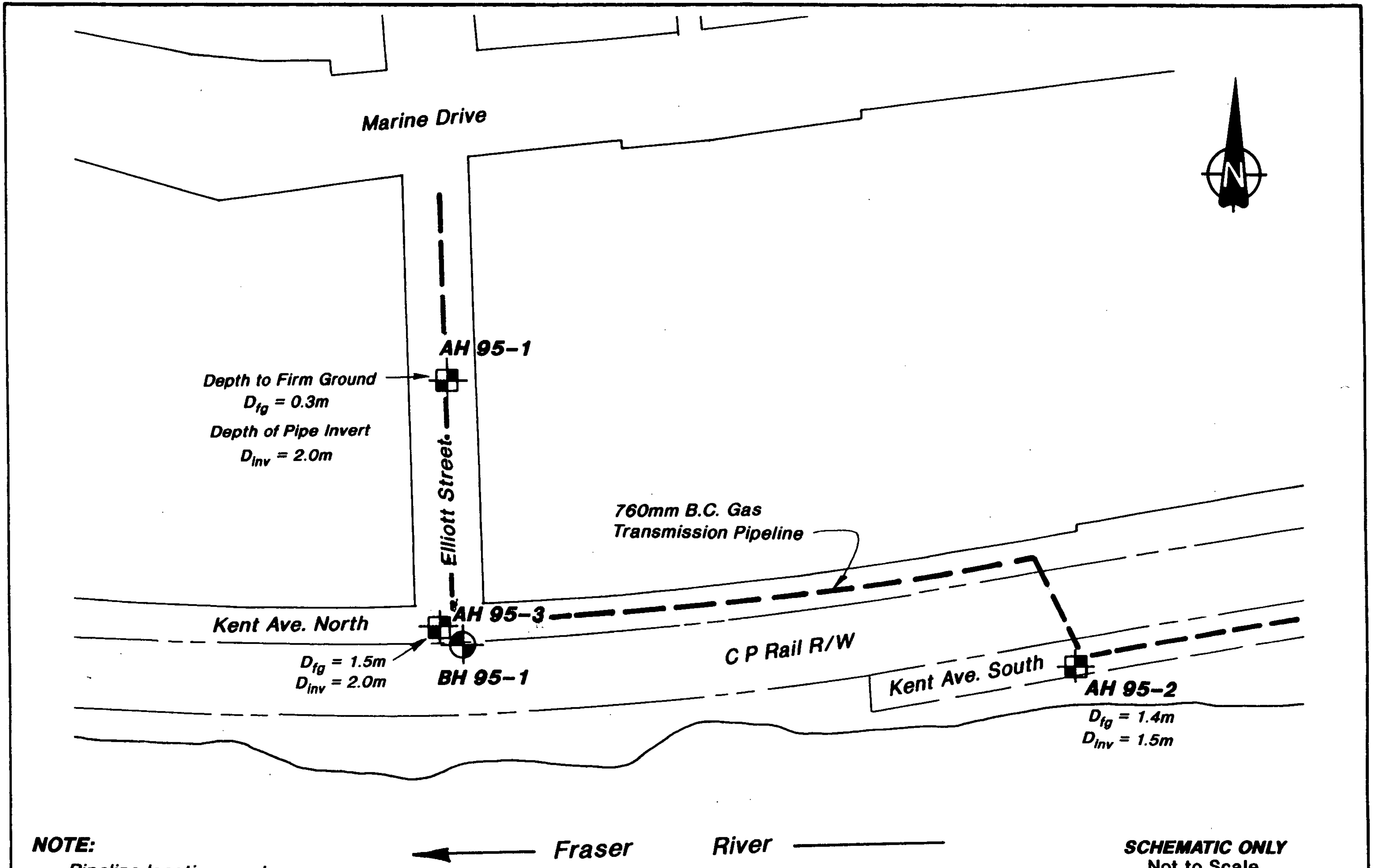
NOTE:
Pipeline locations and alignments schematic only.

Golder Associates

SITE PLAN
FRASER GATE STATION

Figure 2

Project No. 942-7211A Drawn M.T. Reviewed D.W. Date June '95



NOTE:
Pipeline locations and alignments schematic only.

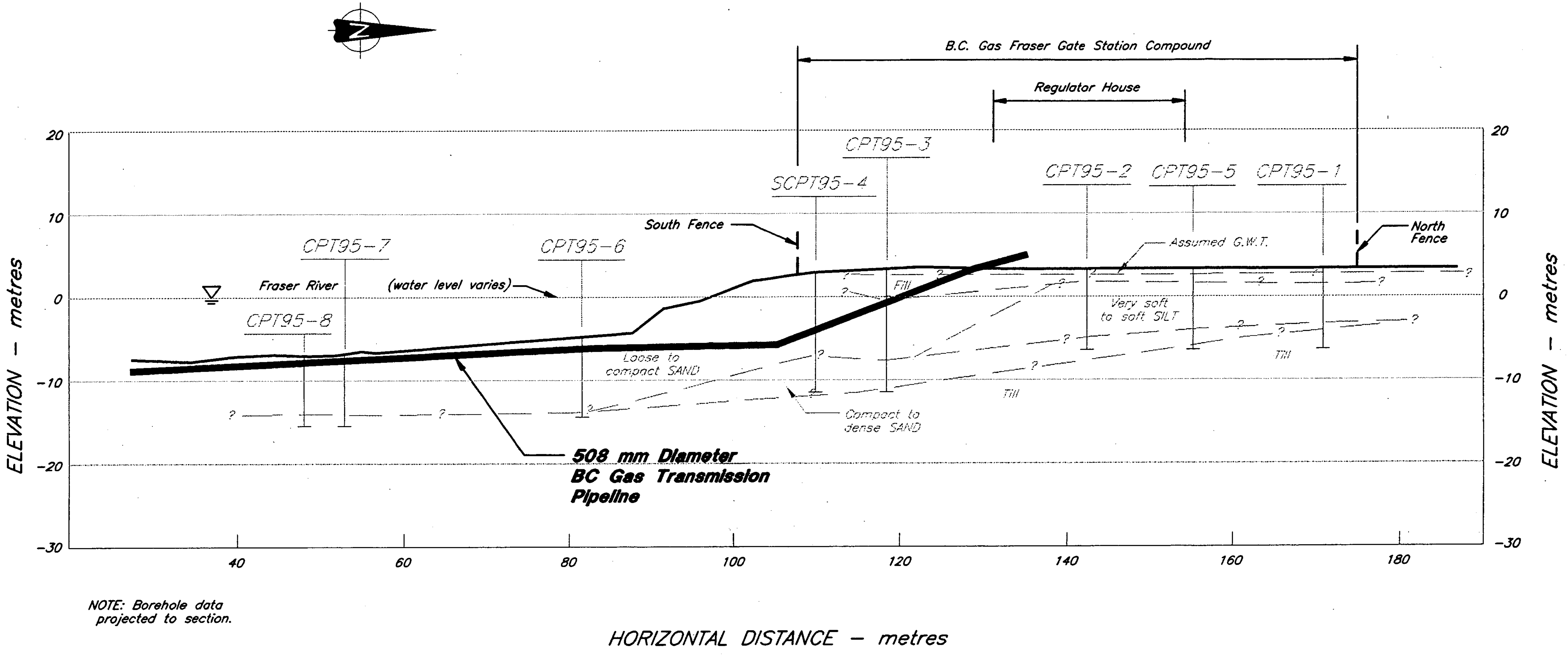
SCHEMATIC ONLY
Not to Scale

Project No. 942-1211A
Drawn w.t.
Reviewed D.W.
Date July 1995

Golder Associates

SITE PLAN
OUTLET PIPELINE AT KENT AVENUE

Figure
3



NOTE: Borehole data
projected to section.

SECTION $\frac{1}{2}$

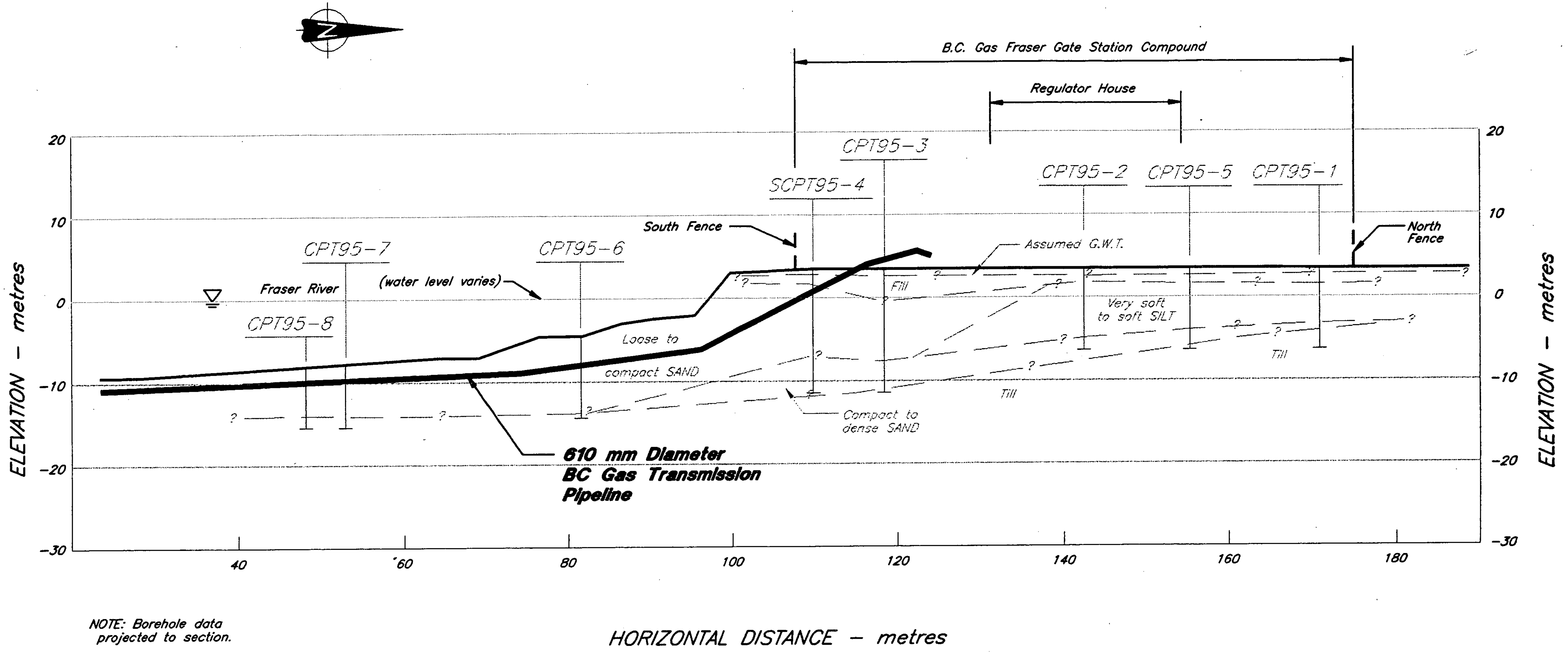
Golder Associates

PROFILE OF EXISTING 508 mm PIPELINE AT FRASER GATE STATION

Figure

4

Project: 942-1211A Drawn: GG Reviewed: D.W. Rev.: JUL-21-95 J:\9421211A\S1211-02.DWG



NOTE: Borehole data projected to section.

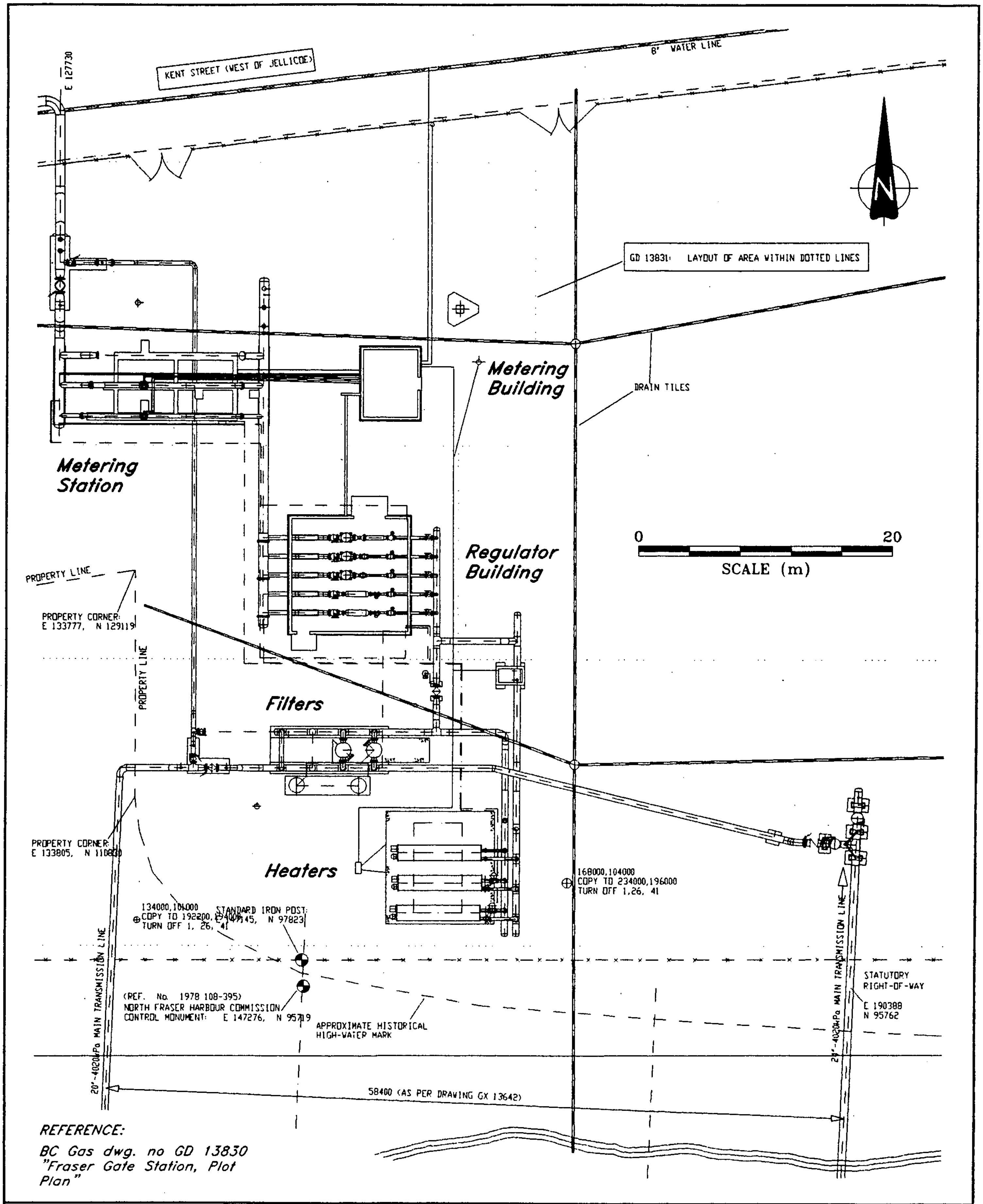
SECTION $\frac{2}{2}$

Golder Associates

PROFILE OF EXISTING 610 mm PIPELINE AT FRASER GATE STATION

Figure
5

Project: 942-1211A Drawn: GG Reviewed: D.W. Rev.: JUL-21-95 J:\9421211A\S1211-01.DWG



REFERENCE:

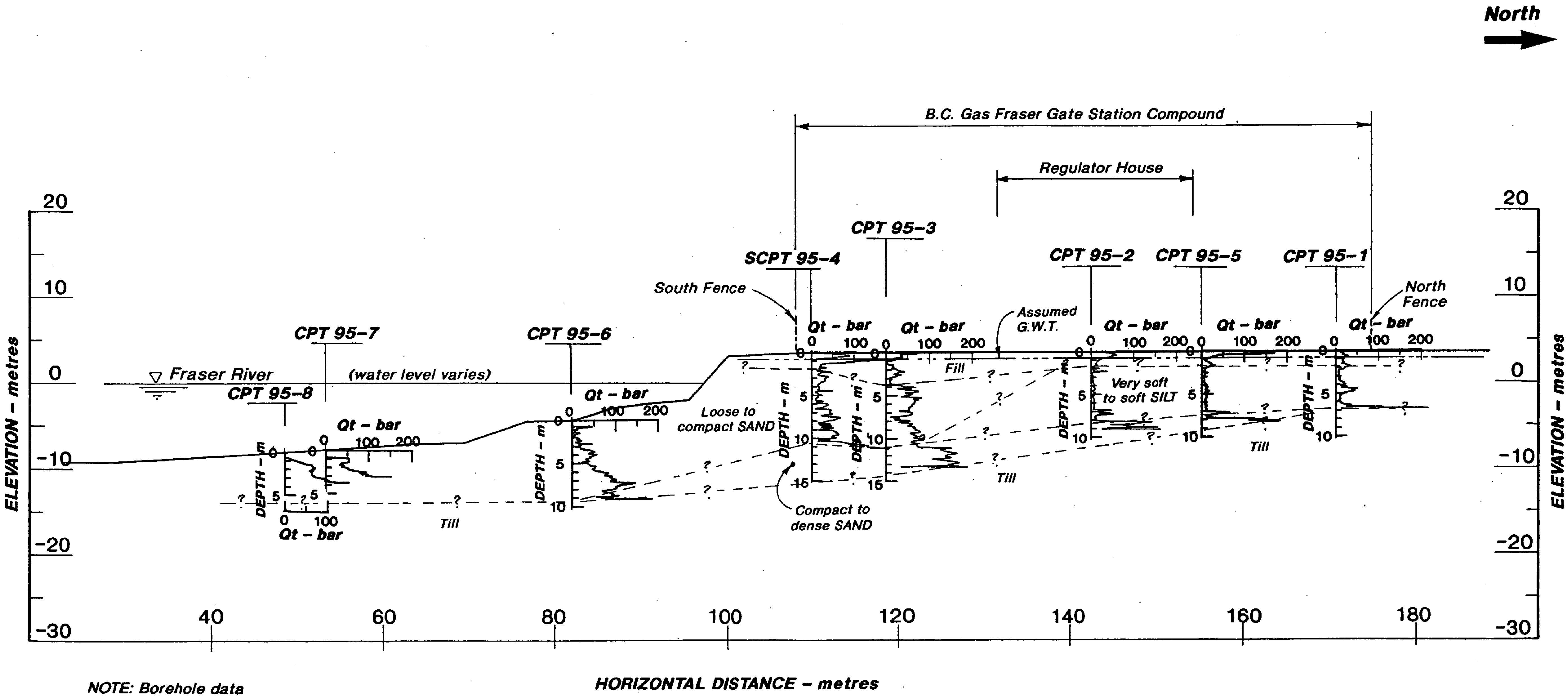
BC Gas dwg. no GD 13830 "Fraser Gate Station, Plot Plan"

Golder Associates

BC GAS PIPING DETAILS
FRASER GATE STATION

Figure 6

Project: 942-1211A Drawn: GG Reviewed: D.W Rev.: JUL-20-95 J:\9421211A\P1211-03.DWG



NOTE: Borehole data
projected to section.

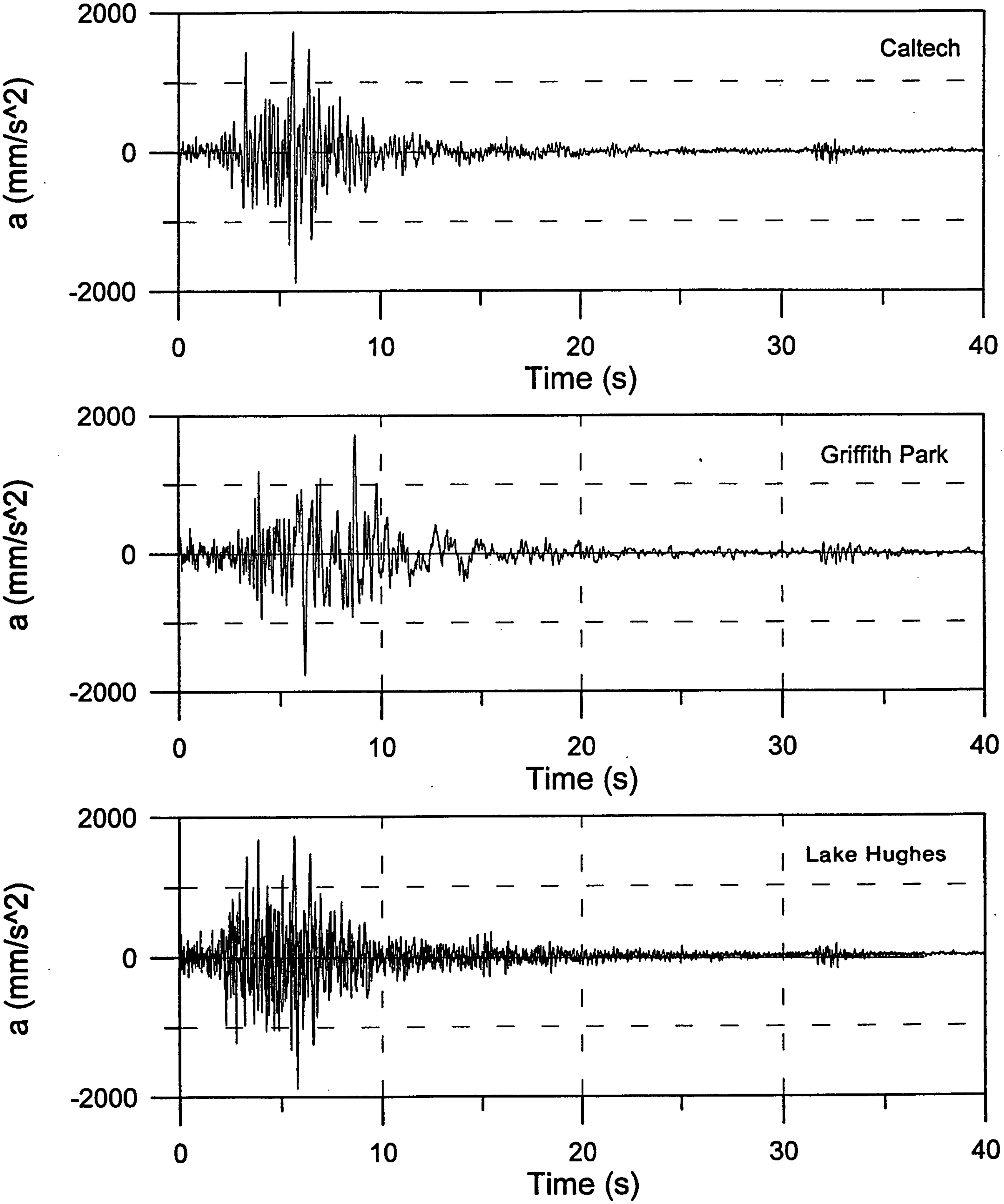
Scale 1 : 500

Golder Associates

INFERRED SOIL STRATIGRAPHY AT FRASER GATE STATION

Figure 7

Project No. 942-1211A Drawn W.I. Reviewed D.W. Date July'95



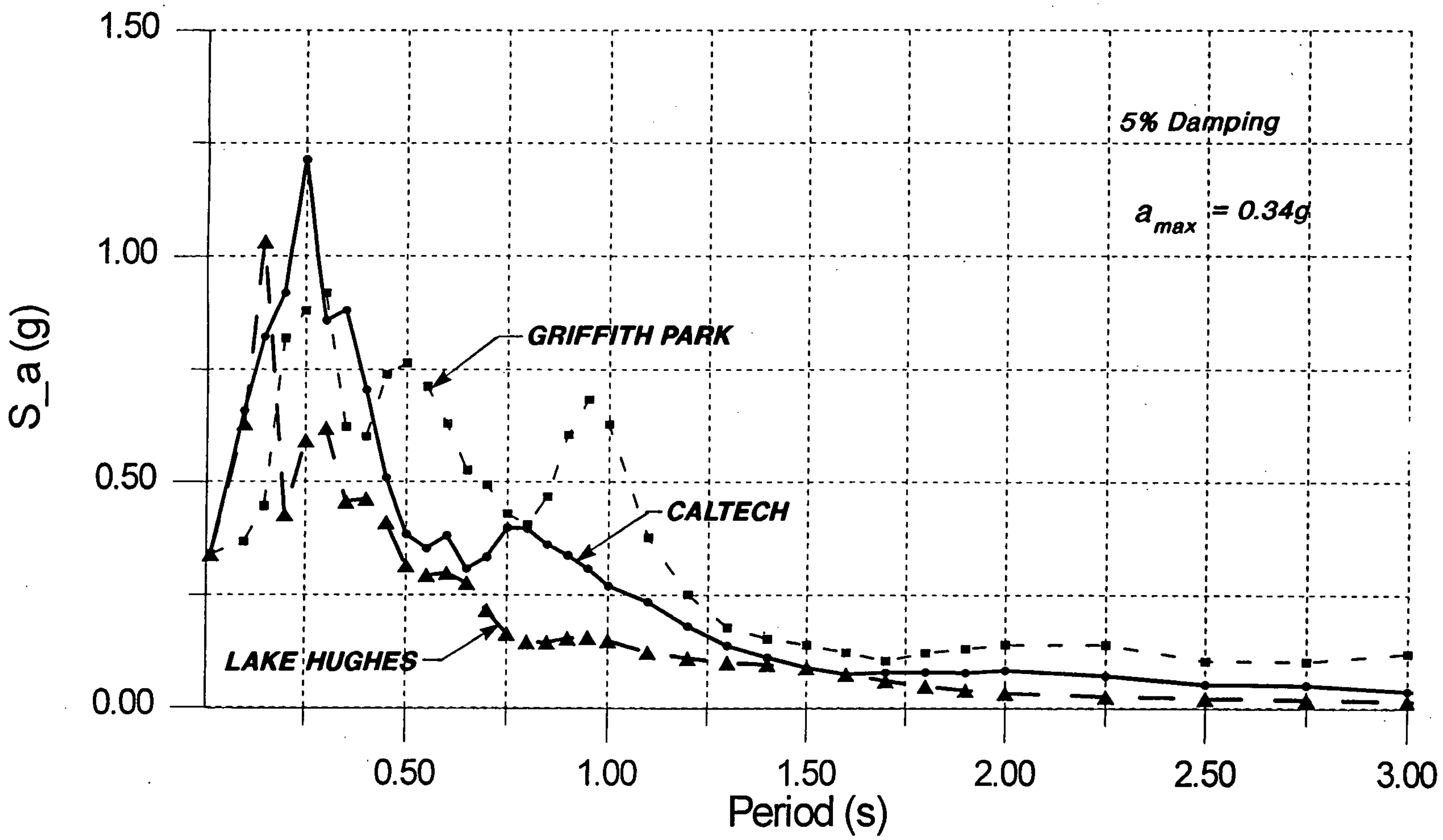
Project No. 942-1211A Drawn BAS Reviewed D.W. Date June 95

Golder Associates

ACCELERATION – TIME HISTORIES USED IN SHAKE ANALYSIS

Figure

8

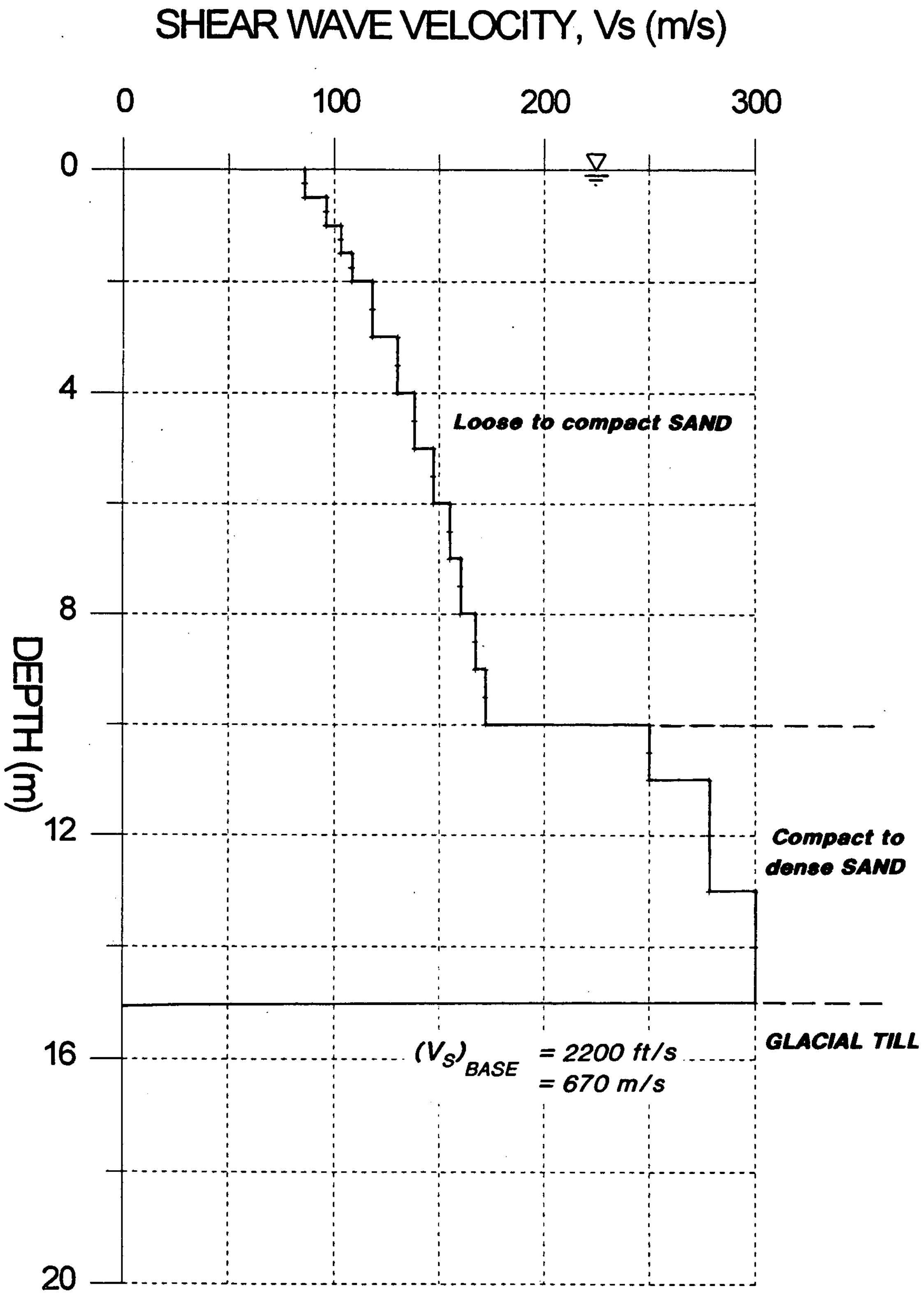


Project No. 942-1211A
Drawn BAD
Reviewed D.W.
Date Feb.'97

Golder Associates

RESPONSE SPECTRA OF EARTHQUAKE GROUND MOTIONS
FROM 1971 SAN FERNANDO EARTHQUAKE
USED IN THE SHAKE ANALYSES

Figure
9

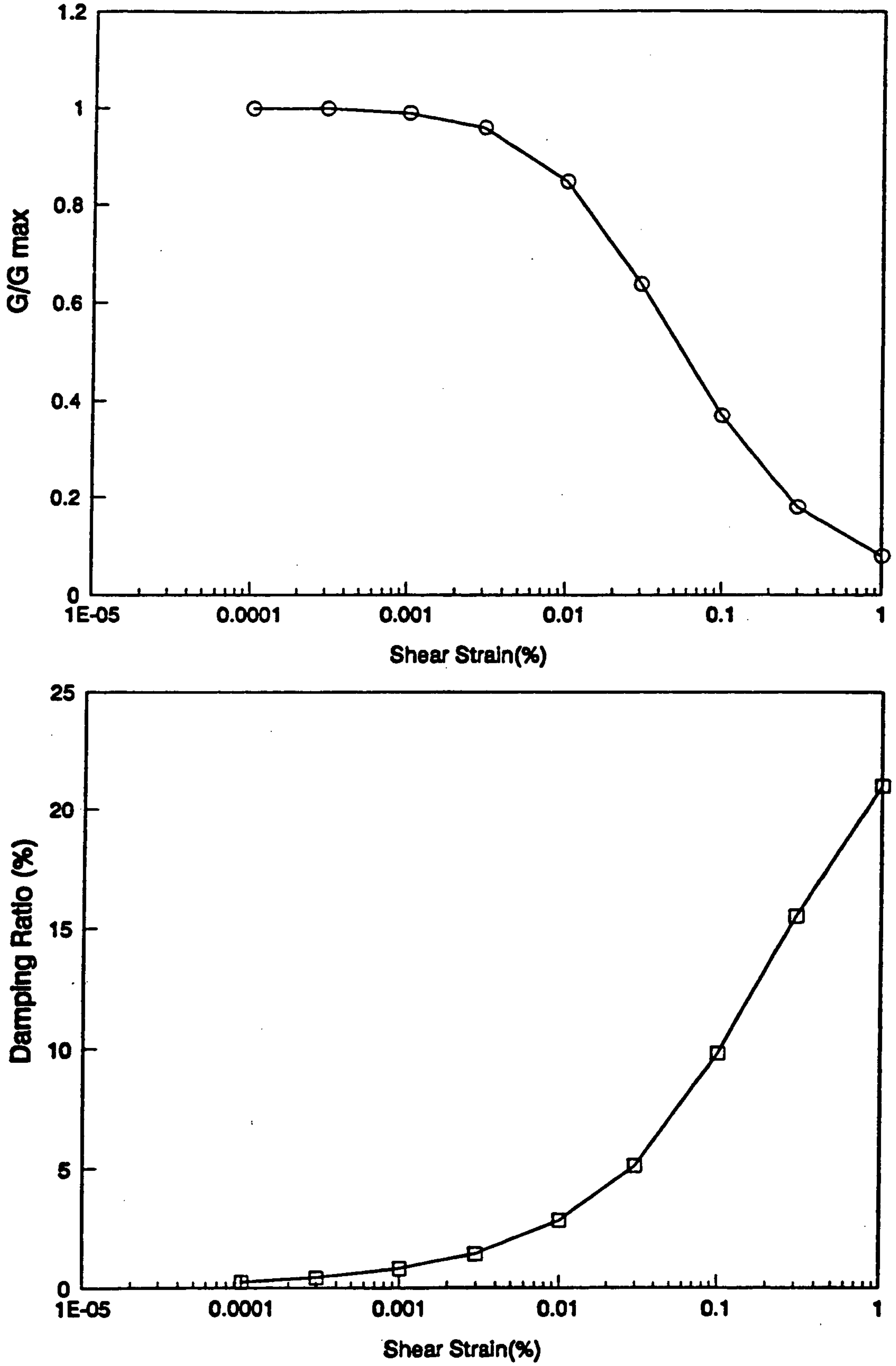


Project No. 942-1211A ...... Drawn BAD ...... Reviewed D.W. ...... Date Feb. 97

Golder Associates

SOIL LAYERING, SHEAR WAVE VELOCITY, & GROUND WATER CONDITIONS ASSUMED IN SHAKE ANALYSES

Figure **10**



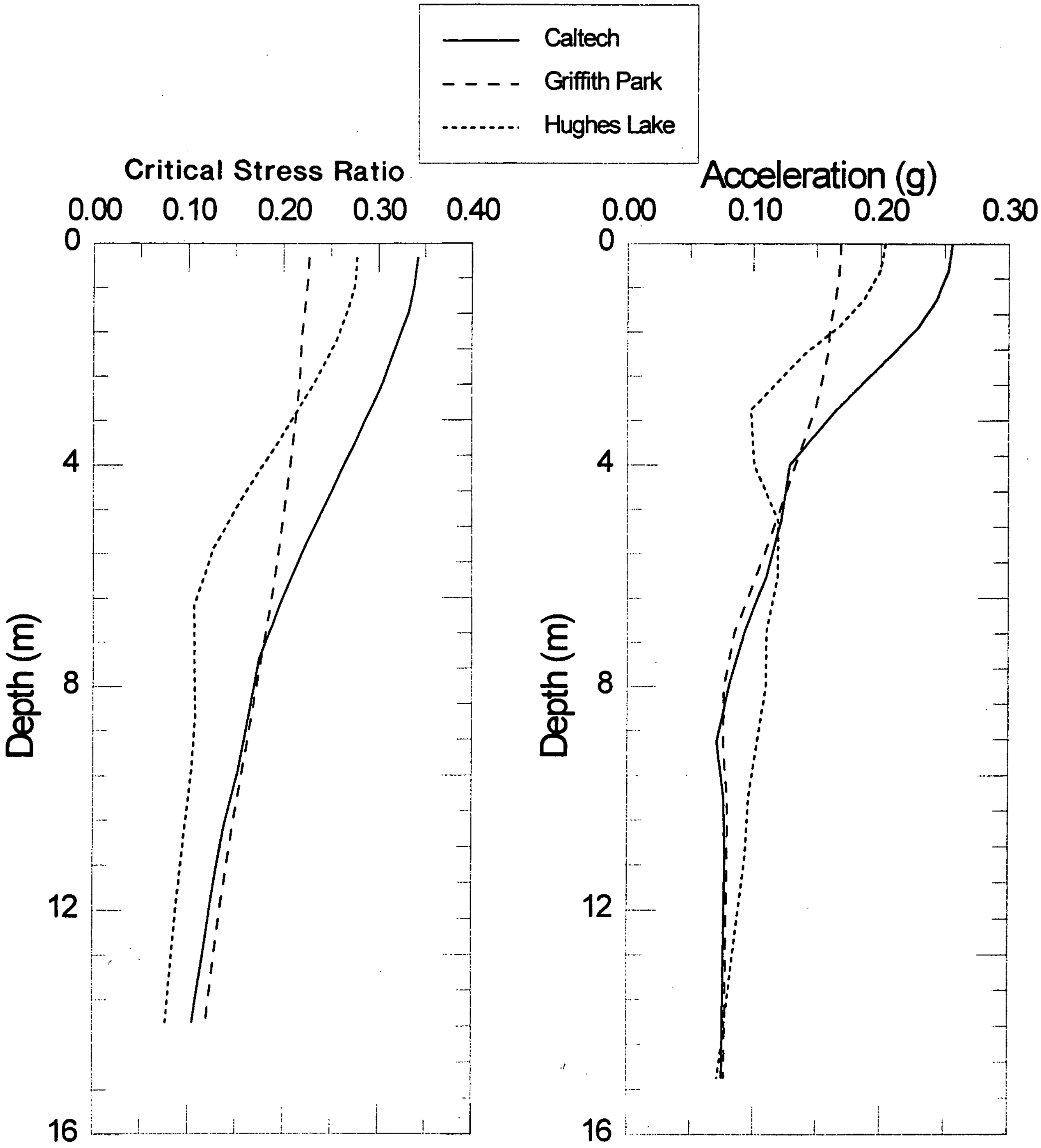
Project No. ...1211A Drawn ...BAC... Reviewed ...D.W.... Date ...June 95

Golder Associates

SHEAR MODULUS AND DAMPING VALUES USED IN SHAKE ANALYSIS

Figure

11

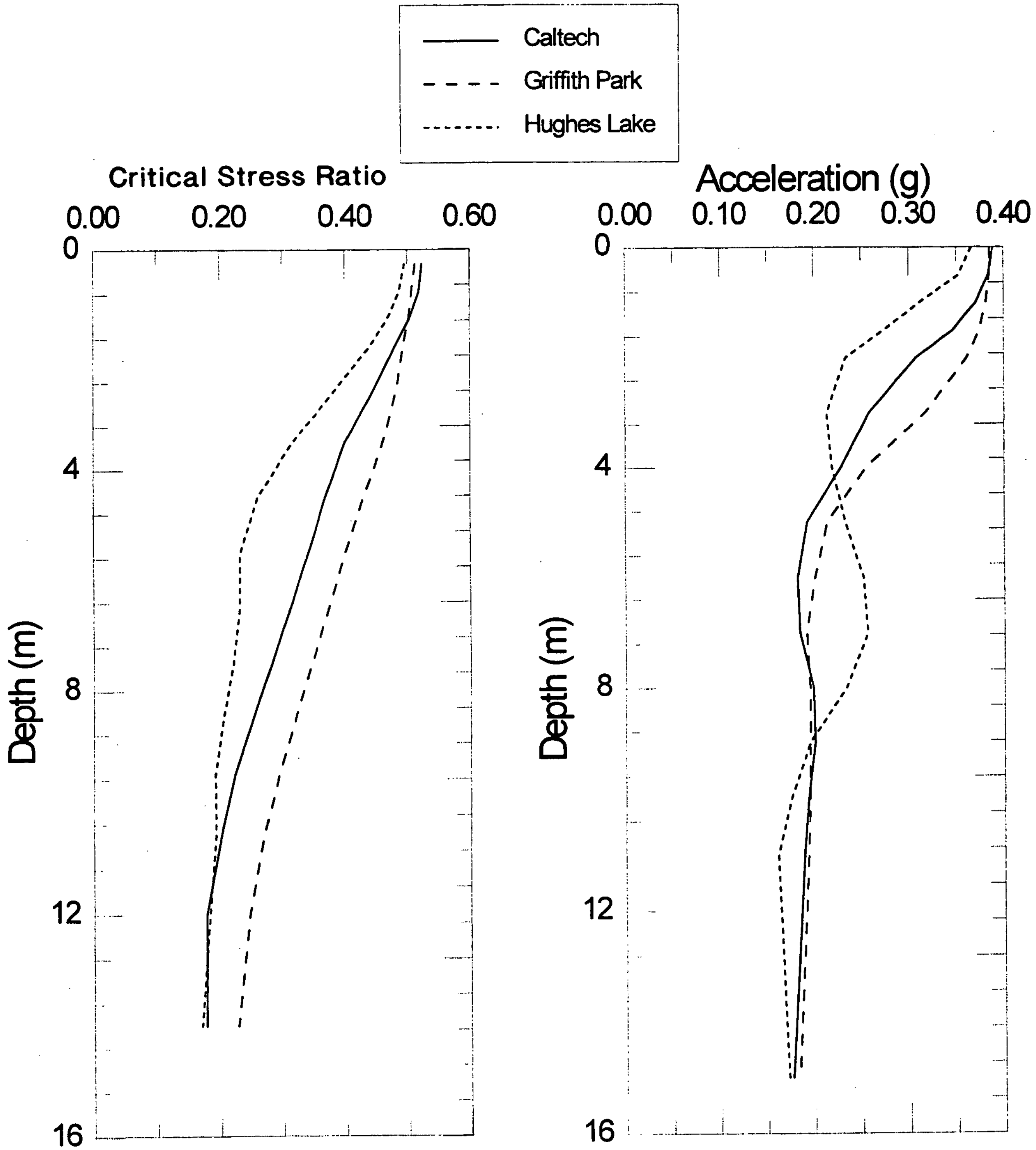


Project No. ...942-1211A... Drawn ...W.T.... Reviewed ...D.W.... Date ...July'95...

Golder Associates

COMPUTED CYCLIC STRESS RATIO and ACCELERATION FROM SHAKE ANALYSES
1 : 100 Year Return Period

Figure 12

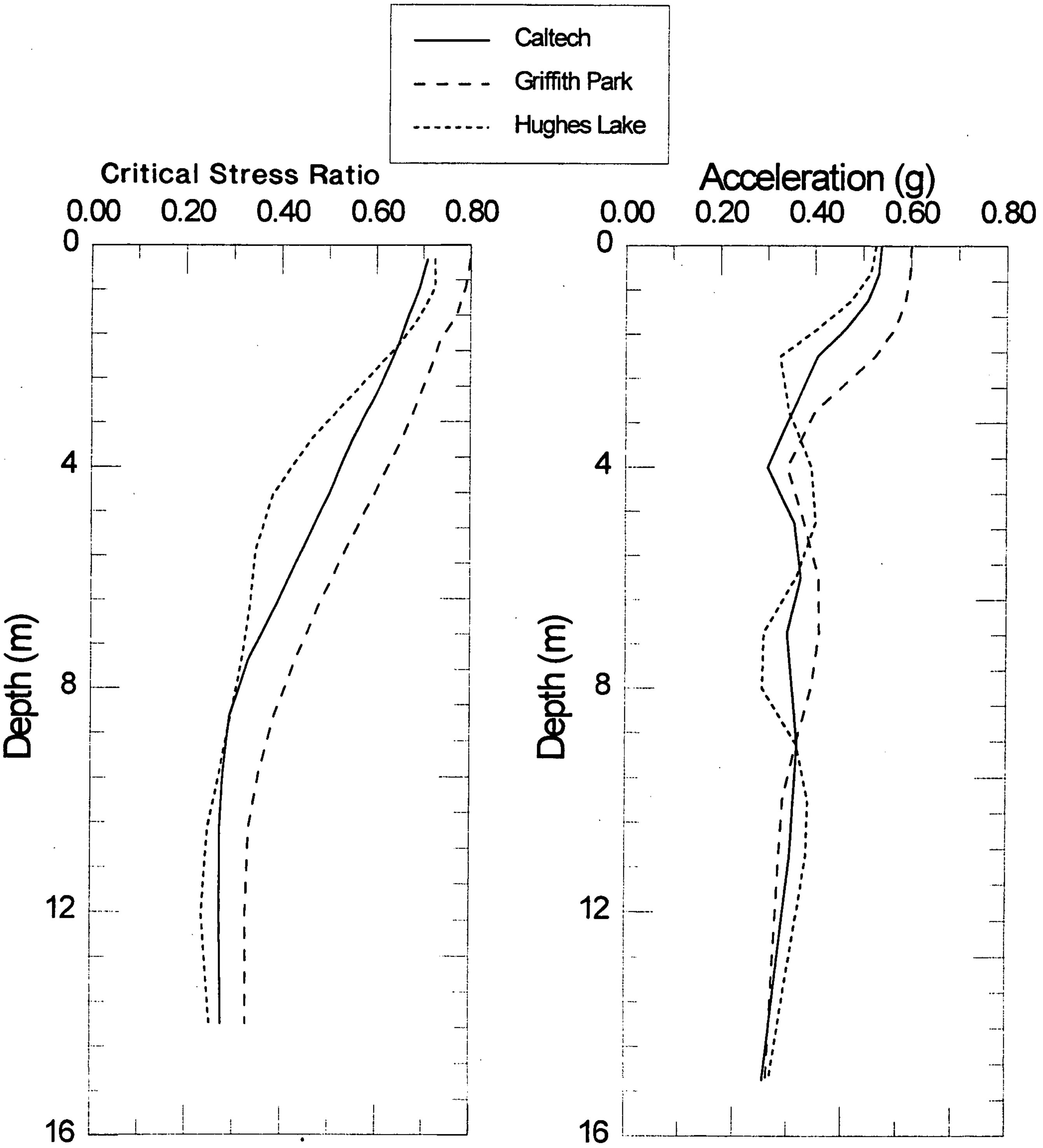


Project No. ...942-1211A... Drawn ...w.t.... Reviewed ...D.W.... Date ...July'95...

Golder Associates

COMPUTED CYCLIC STRESS RATIO and ACCELERATION FROM SHAKE ANALYSES 1 : 475 Year Return Period

Figure **13**

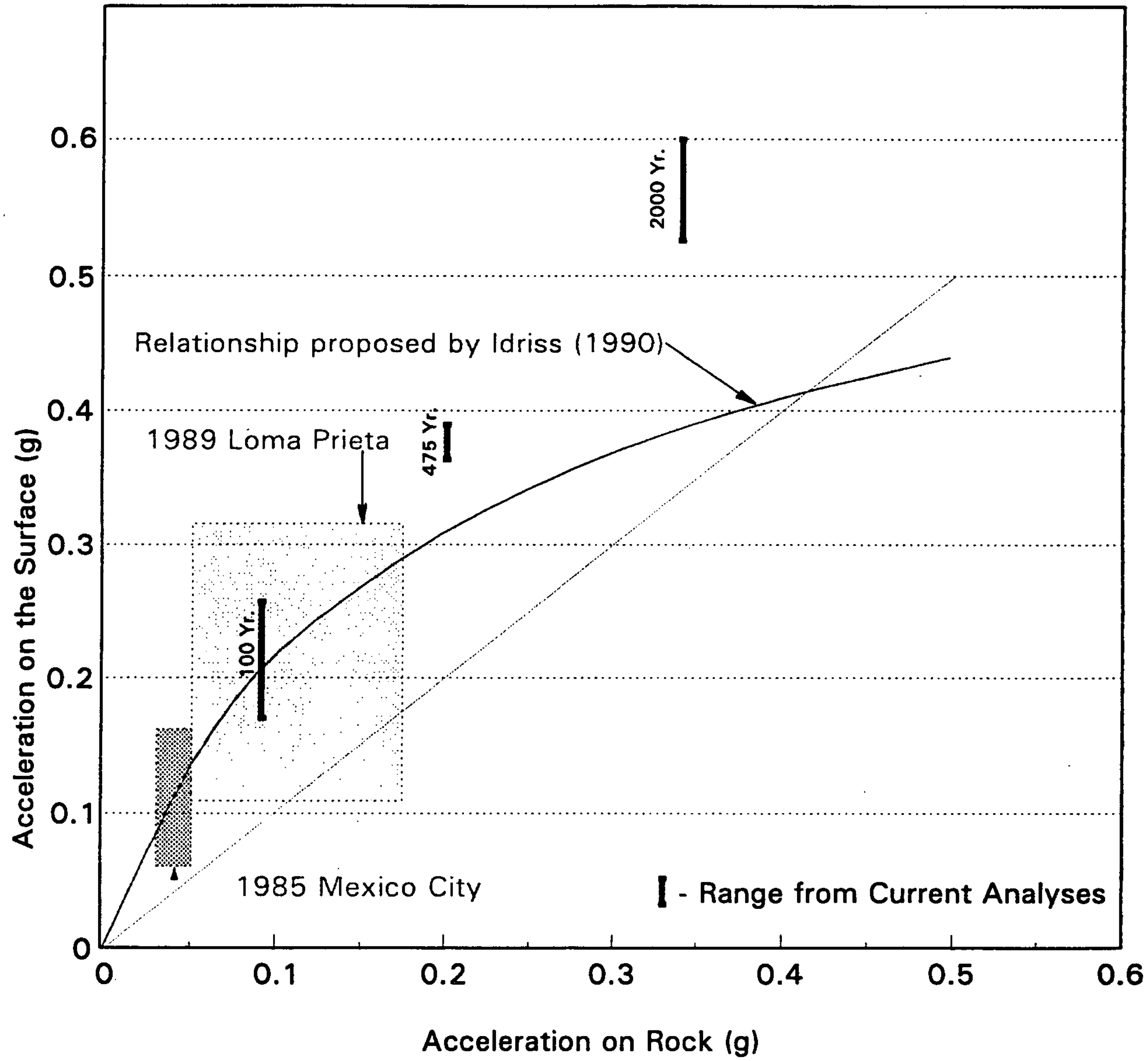


Project No. 942-1211A Drawn w.l. Reviewed D.W. Date July'95

Golder Associates

COMPUTED CYCLIC STRESS RATIO and ACCELERATION FROM SHAKE ANALYSES 1 : 2000 Year Return Period

Figure **14**

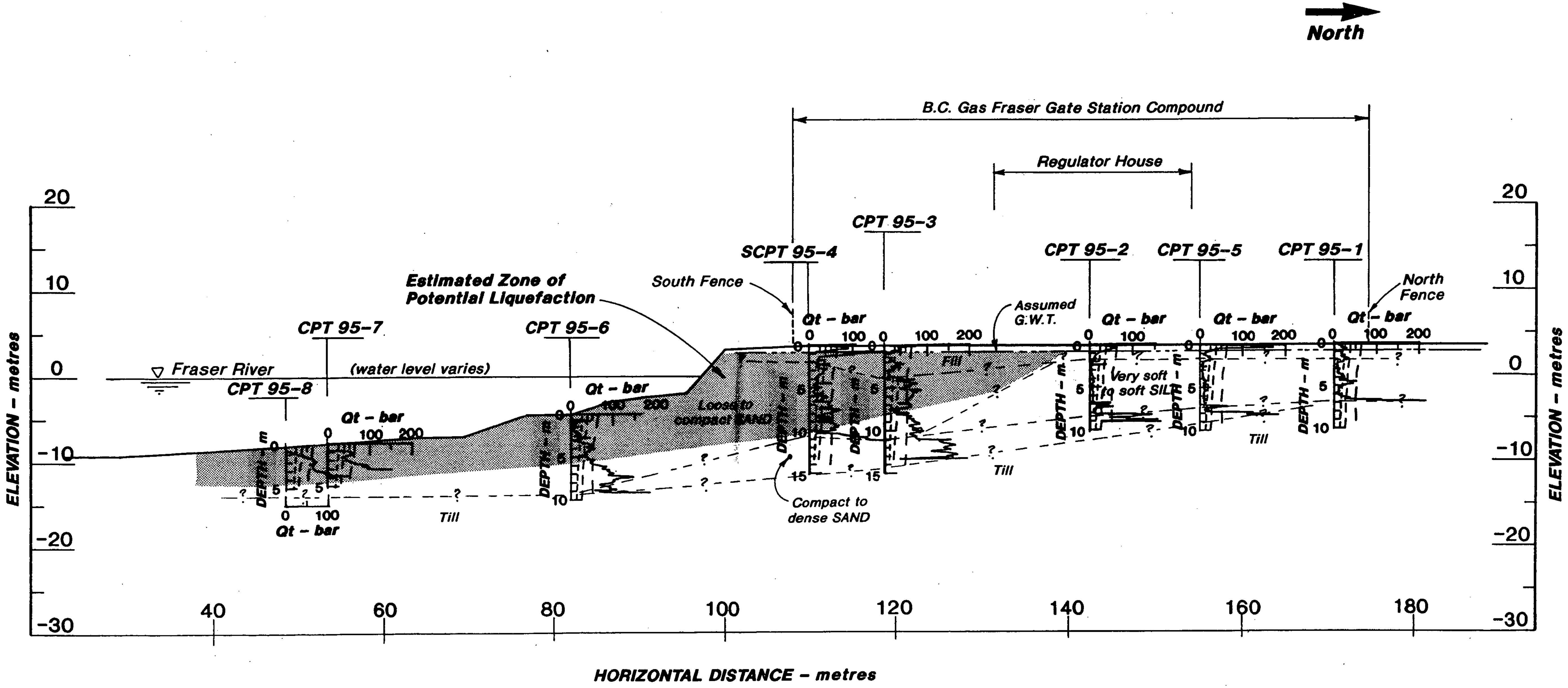


Project No. ....942-1271A.... Drawn ....W.T.... Reviewed ....D.W.... Date ....July/95....

Golder Associates

COMPUTED PEAK GROUND ACCELERATION FROM SHAKE ANALYSES

Figure 15



NOTE: Borehole data projected to section.

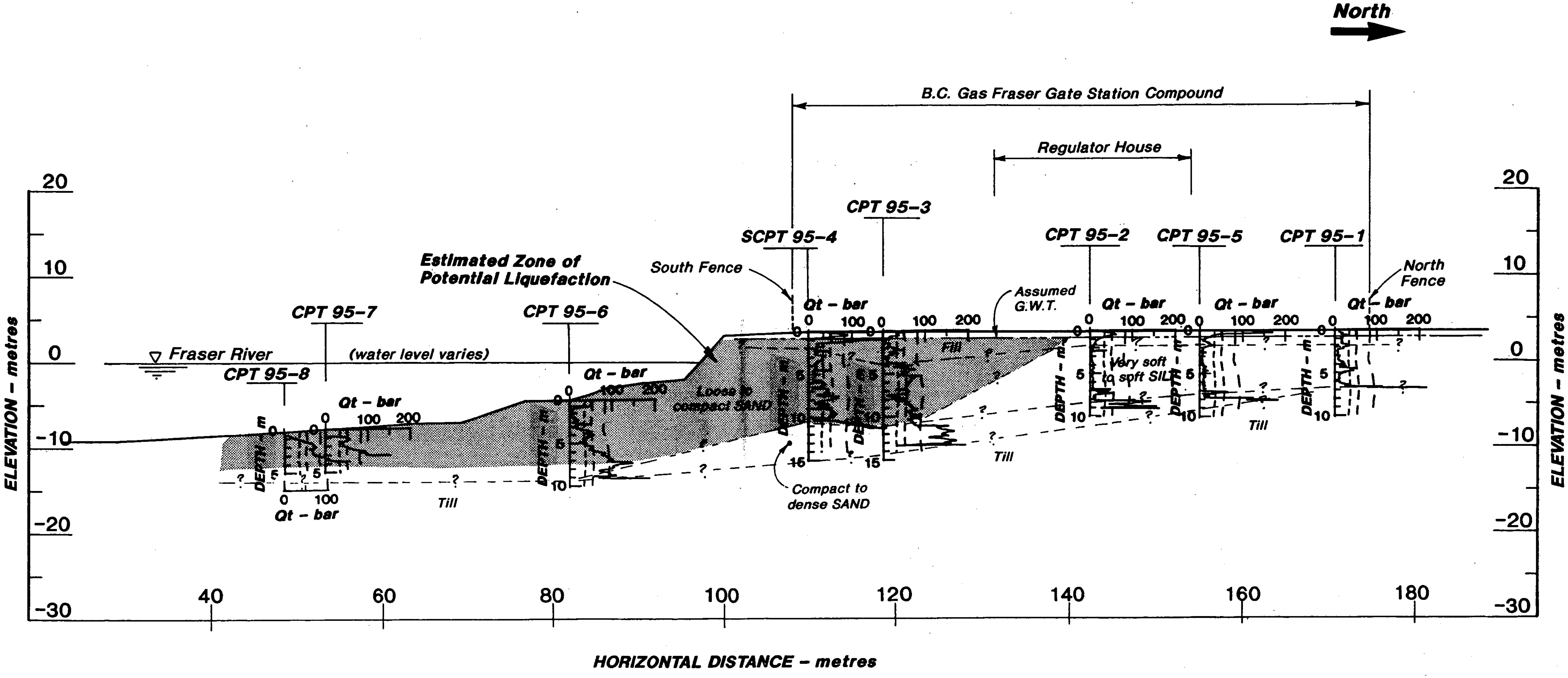
Scale 1 : 500

Project No. 942-1211A Drawn W.T. Reviewed D.W. Date July'95

Golder Associates

ESTIMATED ZONE OF POTENTIAL LIQUEFACTION - 1 : 100 YR. RISK LEVEL

Figure 16



NOTE: Borehole data projected to section.

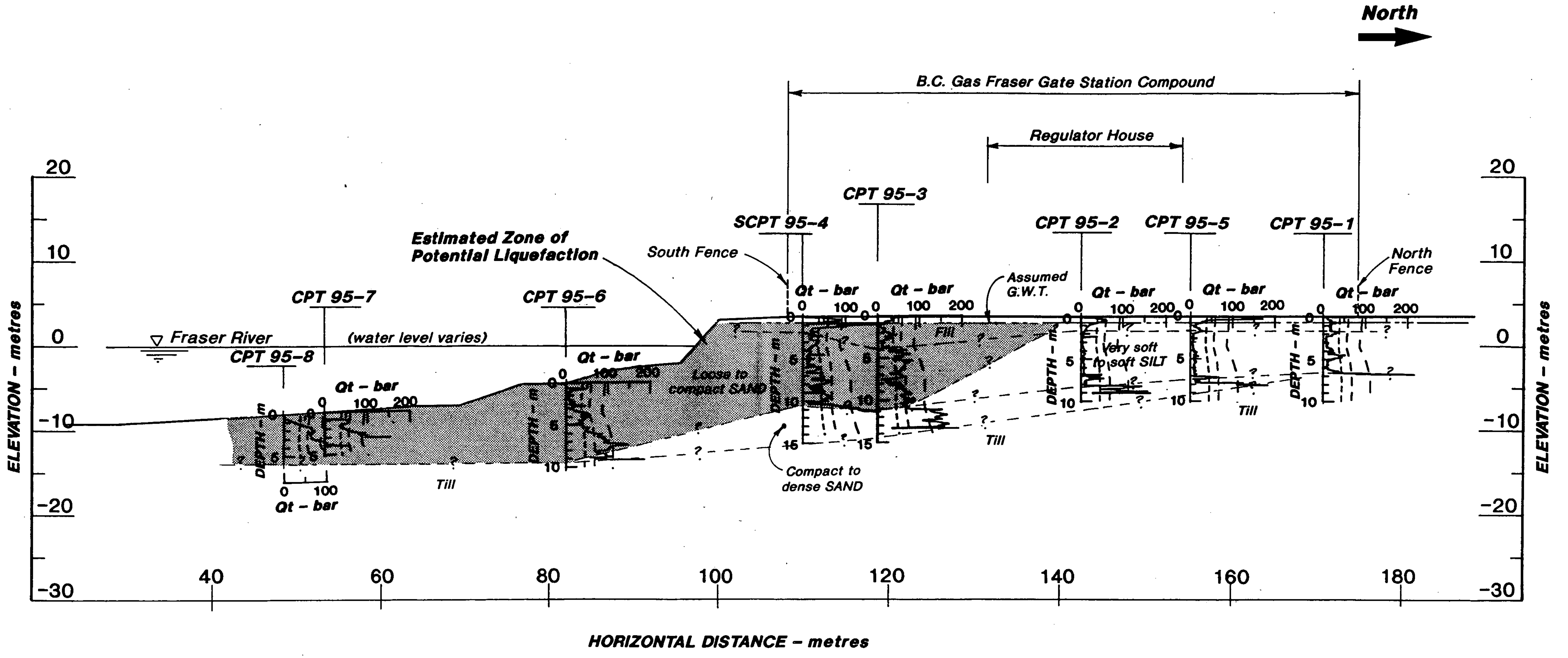
Scale 1 : 500

Golder Associates

ESTIMATED ZONE OF POTENTIAL LIQUEFACTION – 1 : 475 YR. RISK LEVEL

Figure 17

Project No. 942-1211A Drawn w.f. Reviewed D.W. Date July'95



NOTE: Borehole data projected to section.

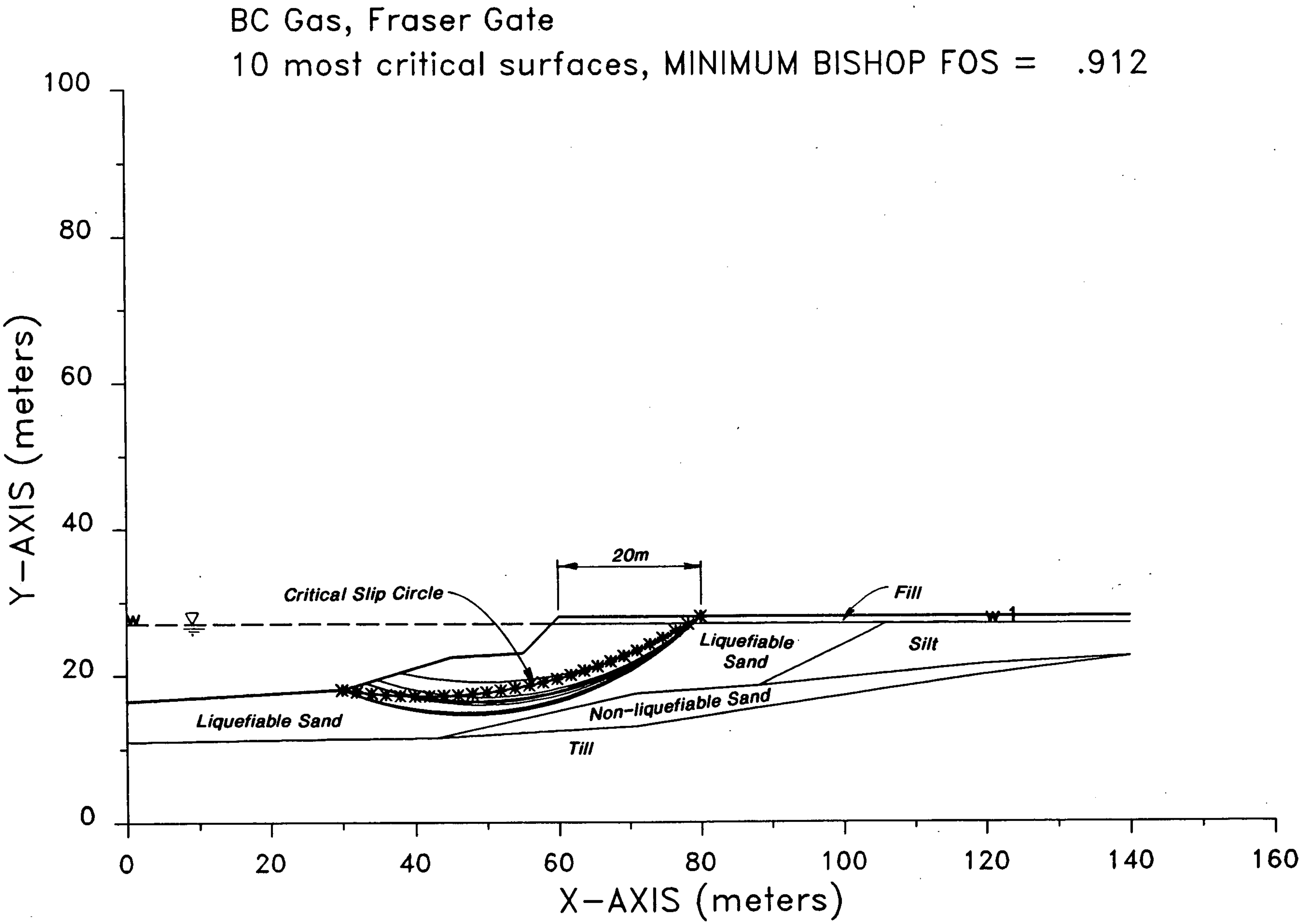
Scale 1 : 500

Golder Associates

ESTIMATED ZONE OF POTENTIAL LIQUEFACTION - 1 : 2000 YR. RISK LEVEL

Figure 18

Project No. 942-1211A Drawn w.f. Reviewed D.W. Date July'95

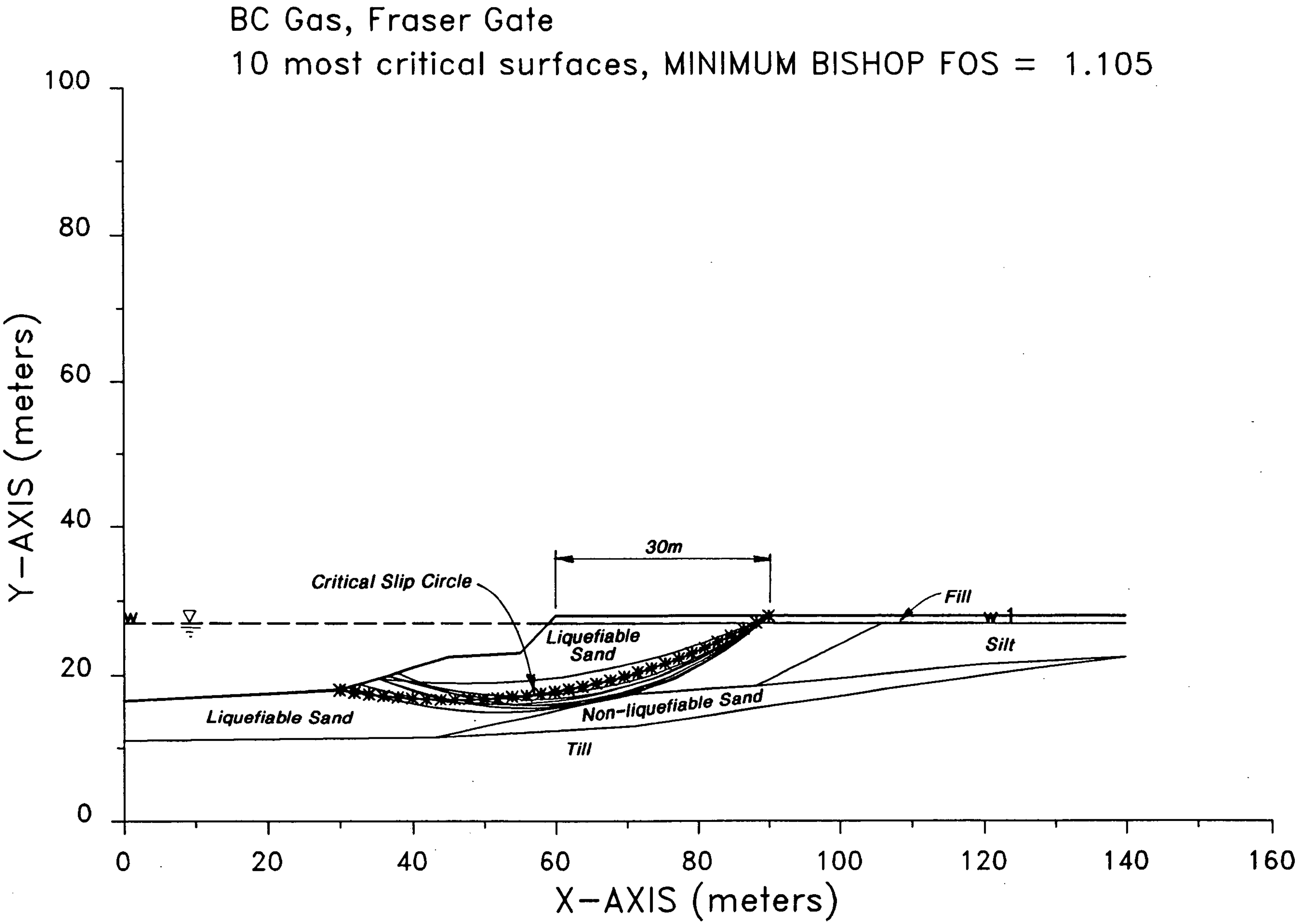


Project No. 942-1211A
Drawn w.l.
Reviewed D.W.
Date July 1995

Golder Associates

SLOPE STABILITY ANALYSIS – FREE FIELD – CASE I
SLIP SURFACE EXTENDING 20m BACK FROM RIVER BANK
(Post-liquefaction, with no Seismic Coefficient)

Figure **19**



Project No. 942-1211A
Drawn w.t.
Reviewed D.W.
Date July 1995

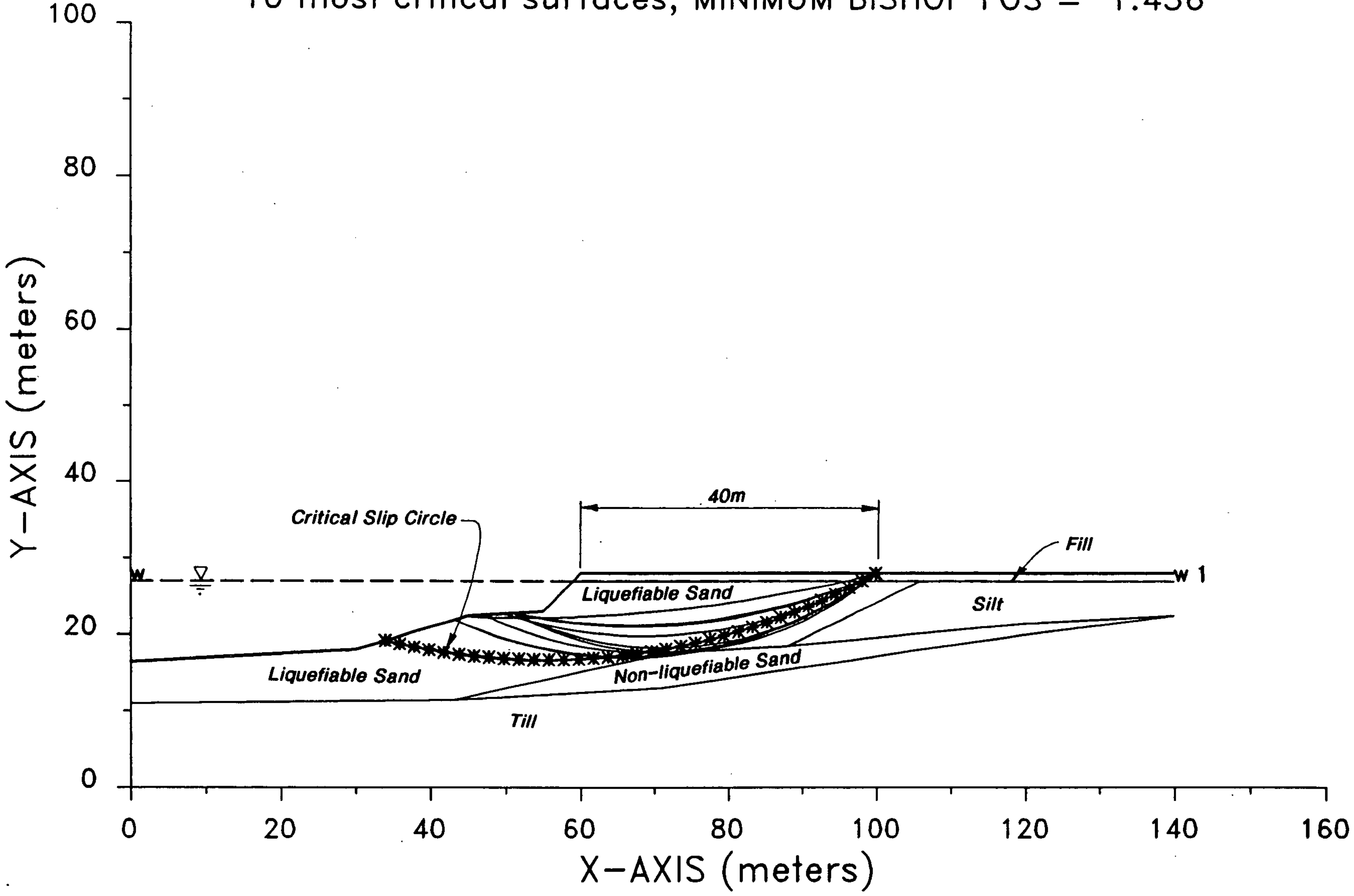
Golder Associates

SLOPE STABILITY ANALYSIS – FREE FIELD – CASE II
SLIP SURFACE EXTENDING 30m BACK FROM RIVER BANK
(Post-liquefaction, with no Seismic Coefficient)

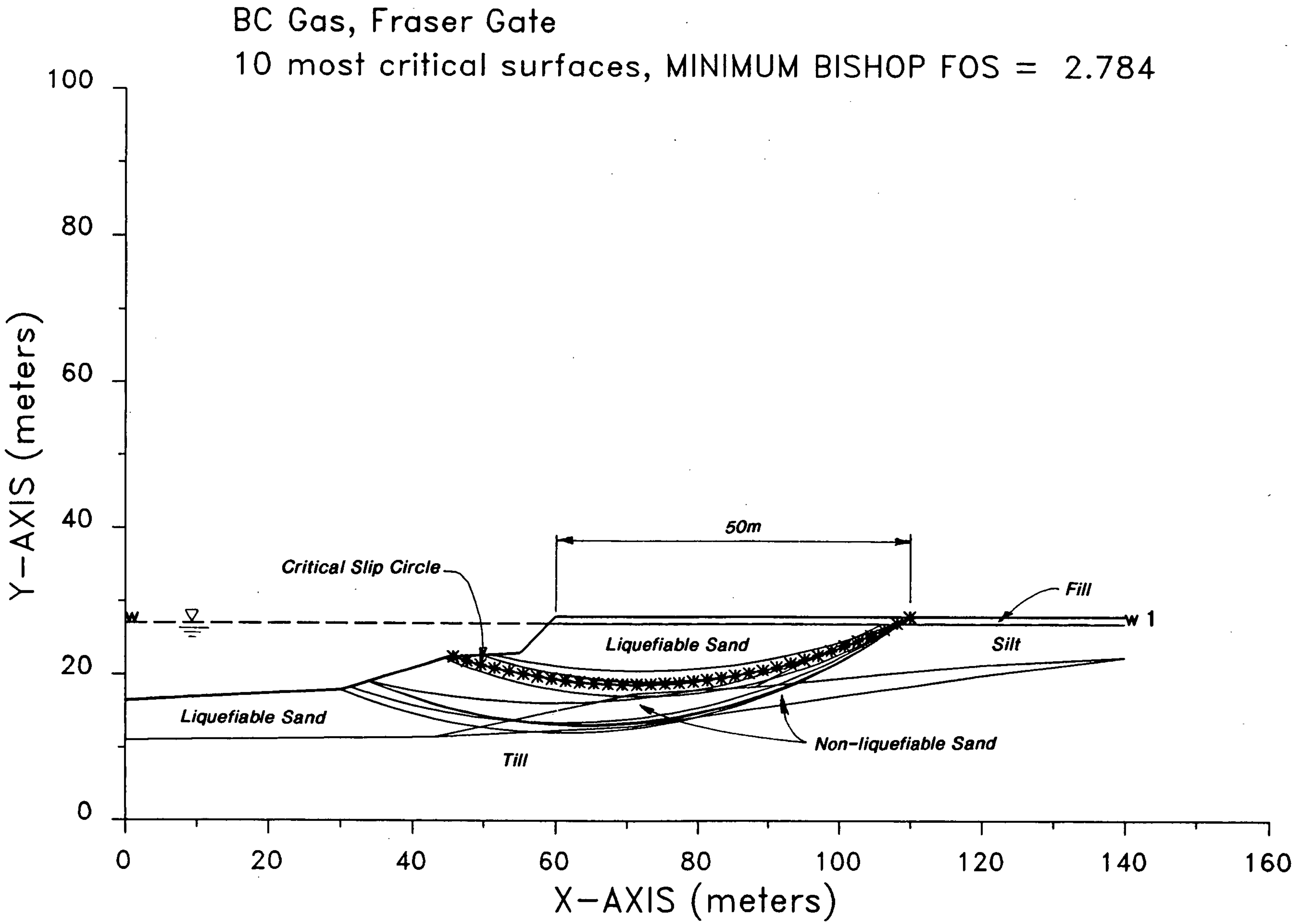
Figure
20

BC Gas, Fraser Gate

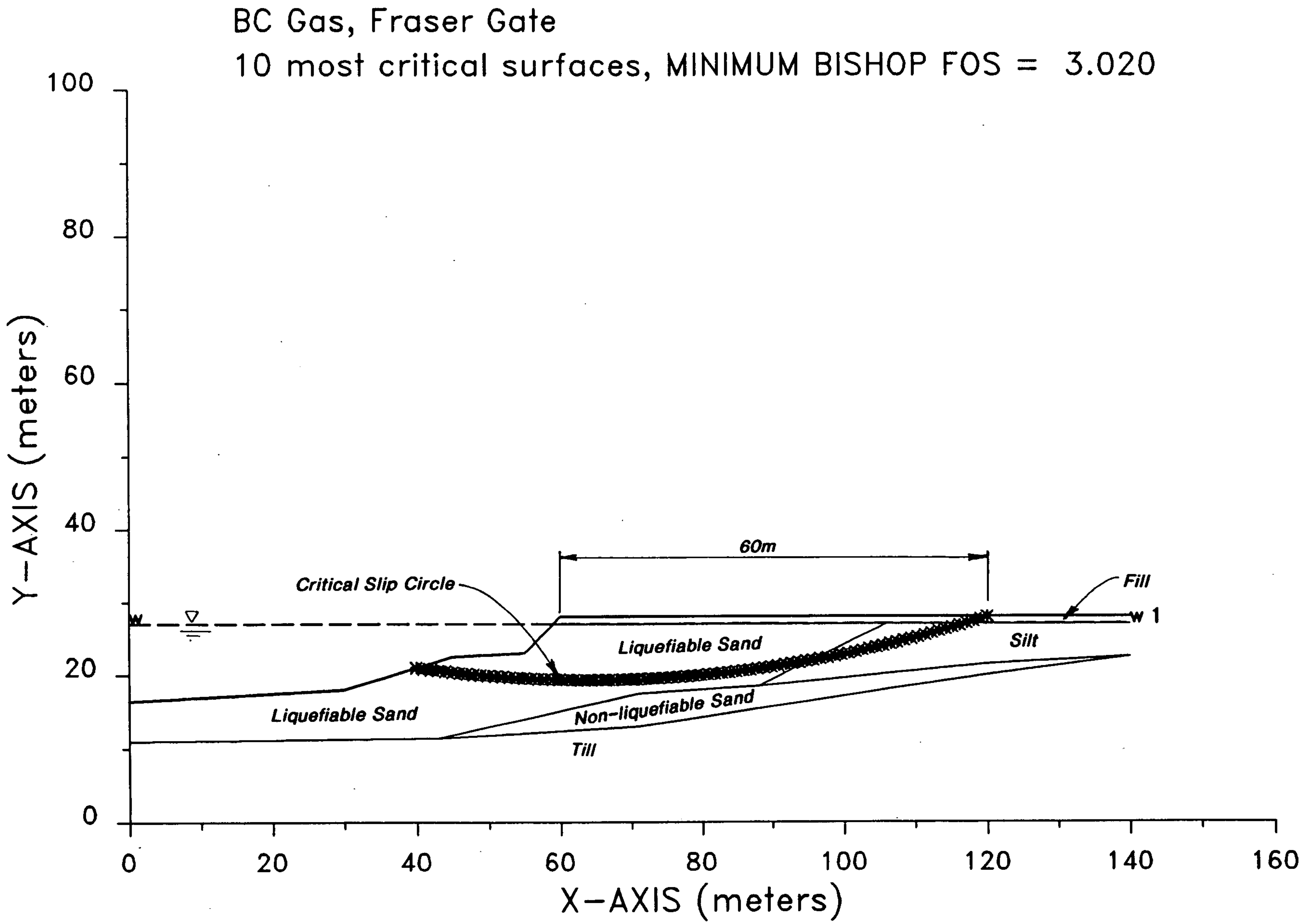
10 most critical surfaces, MINIMUM BISHOP FOS = 1.436



Project No. 942-1211A Drawn w.f. Reviewed D.W. Date July 1995	Golder Associates	**SLOPE STABILITY ANALYSIS - FREE FIELD - CASE III** **SLIP SURFACE EXTENDING 40m BACK FROM RIVER BANK** **(Post-liquefaction, with no Seismic Coefficient)**	Figure **21**



Project No. 942-1211A Drawn w.l. Reviewed D.W. Date July 1995	Golder Associates	**SLOPE STABILITY ANALYSIS – FREE FIELD – CASE IV** **SLIP SURFACE EXTENDING 50m BACK FROM RIVER BANK** **(Post-liquefaction, with no Seismic Coefficient)**	Figure **22**



Project No. 942-1211A
Drawn w.l.
Reviewed D.W.
Date July 1995

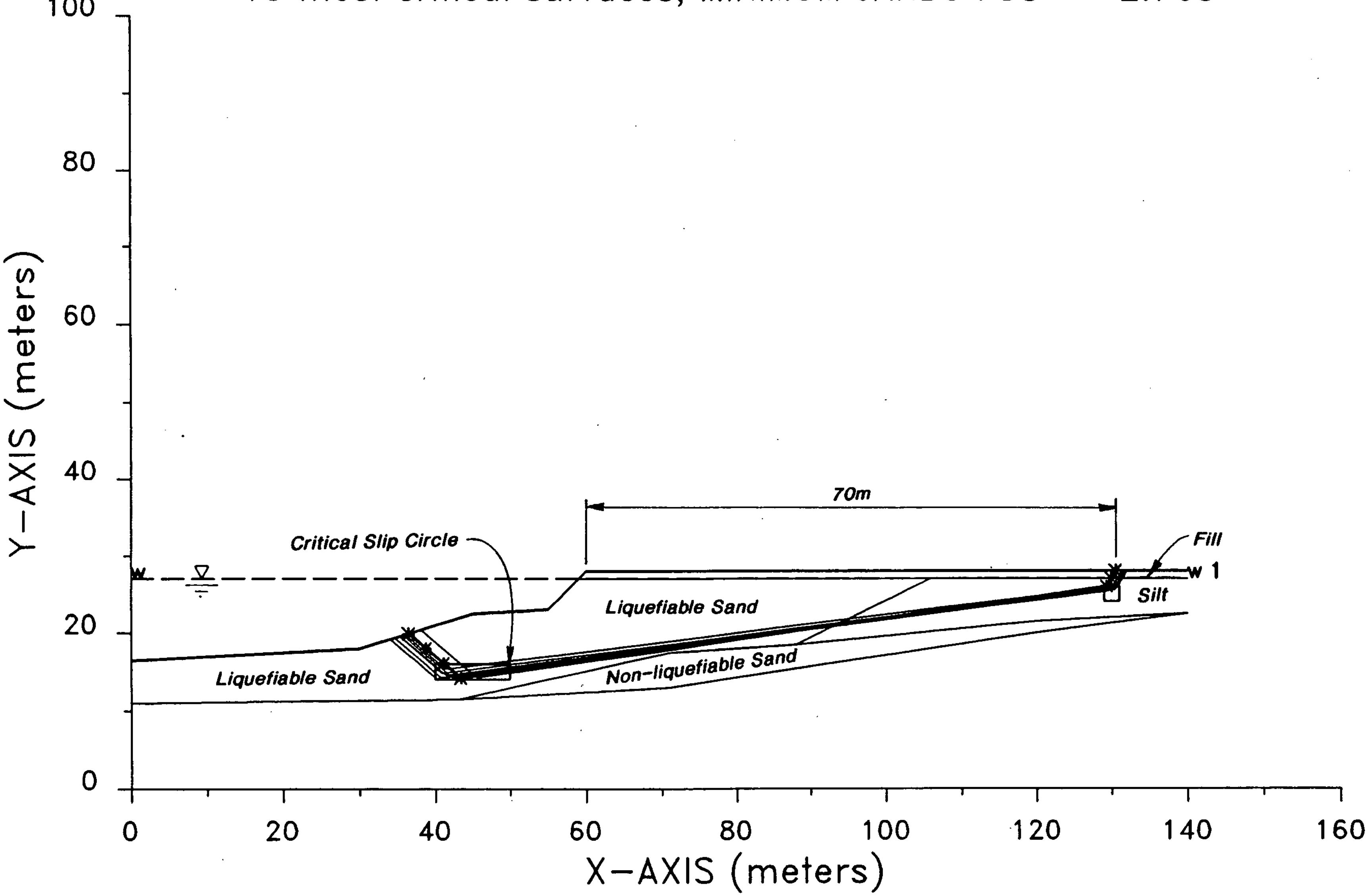
Golder Associates

SLOPE STABILITY ANALYSIS – FREE FIELD – CASE V
SLIP SURFACE EXTENDING 60m BACK FROM RIVER BANK
(Post-liquefaction, with no Seismic Coefficient)

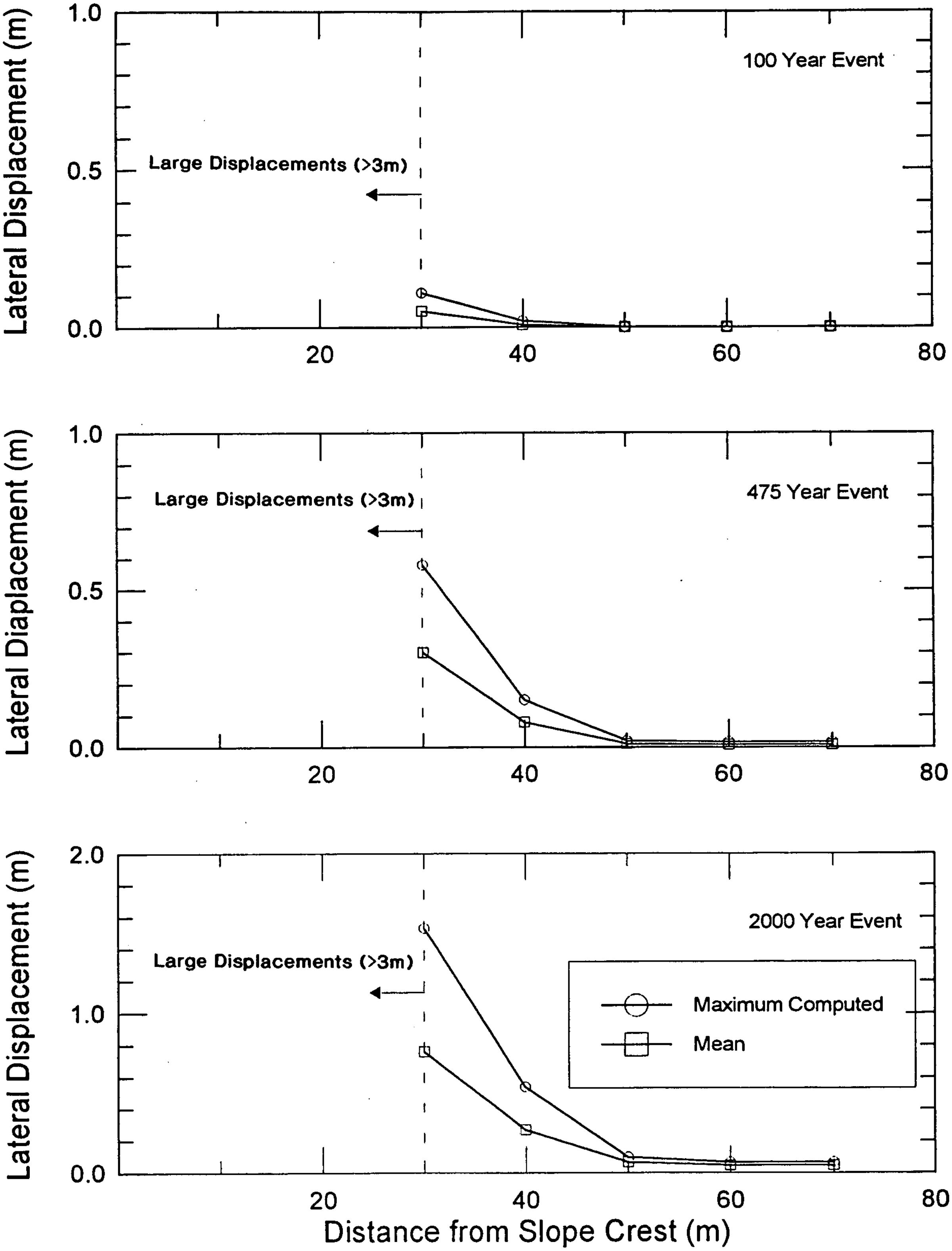
Figure
23

BC Gas, Fraser Gate

10 most critical surfaces, MINIMUM JANBU FOS = 2.765



Project No. 942-1211A Drawn w.t. Reviewed D.W. Date July 1995	Golder Associates	**SLOPE STABILITY ANALYSIS - FREE FIELD - CASE VI** **SLIP SURFACE EXTENDING 70m BACK FROM RIVER BANK** **(Post-liquefaction, with no Seismic Coefficient)**	Figure **24**

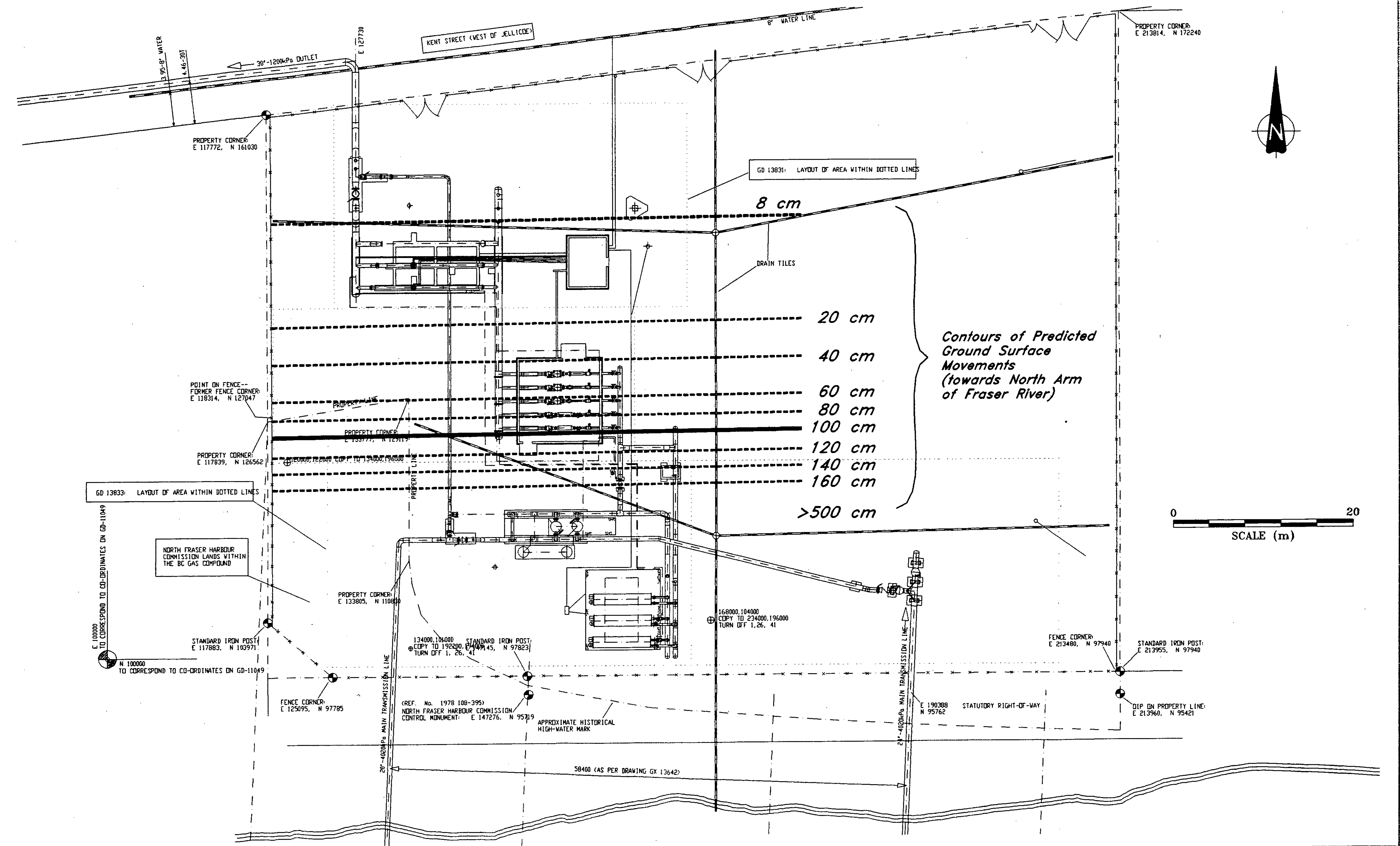


Project No. 942-1211A Drawn W.T. Reviewed D.W. Date July'95

Golder Associates

PREDICTED EARTHQUAKE–INDUCED GROUND DISPLACEMENTS USING DISPLMT PROGRAM

Figure 25



REFERENCE:

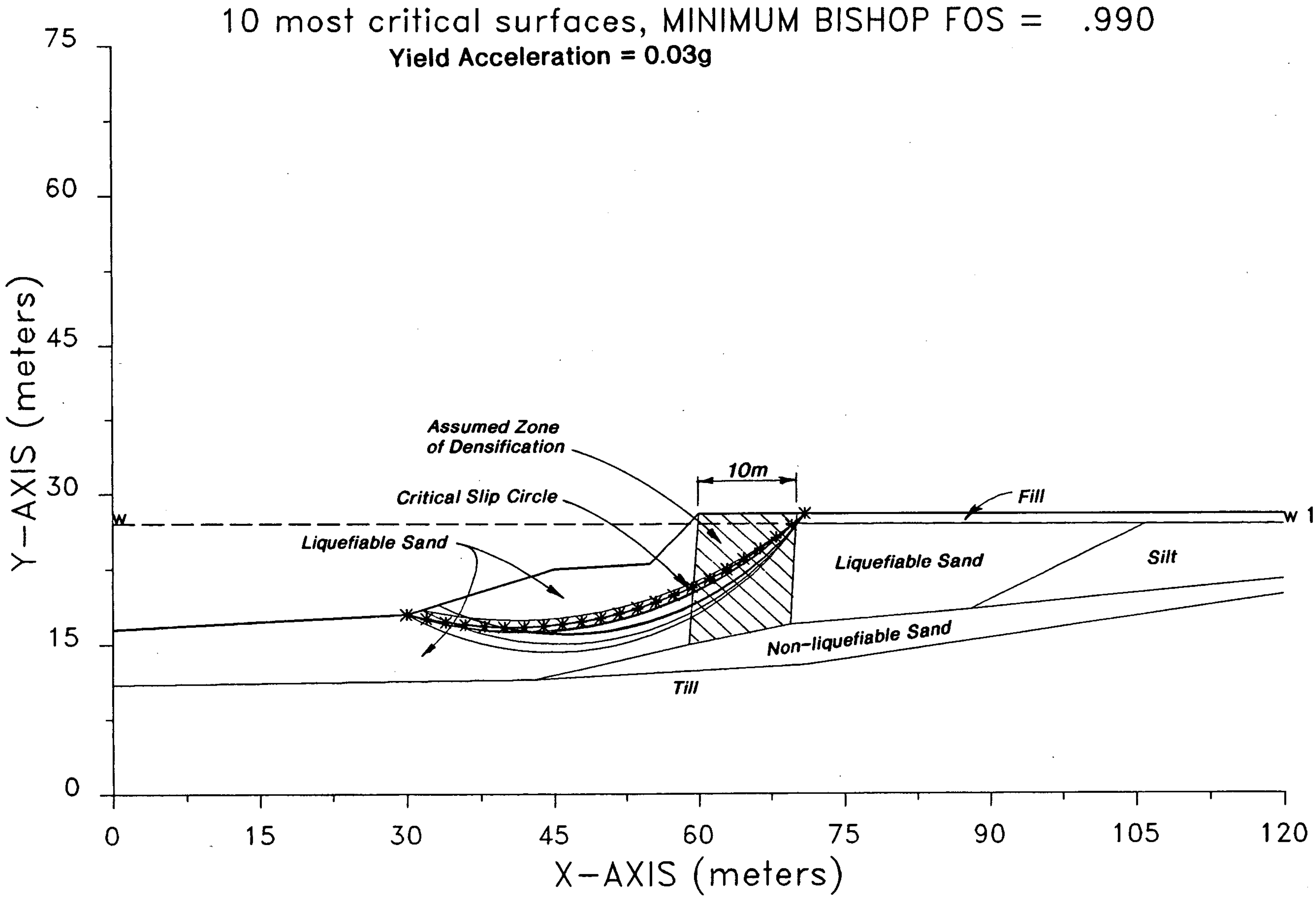
BC Gas dwg. no GD 13830 "Fraser Gate Station, Plot Plan"

Golder Associates

CONTOURS OF PREDICTED LIQUEFACTION-INDUCED GROUND MOVEMENTS
Return Period 1 :2000

Figure **26**

Project: 942-1211A Drawn: GG Reviewed: D.W. Rev.: JUL-20-95 J:\9421211A\P1211-02.DWG

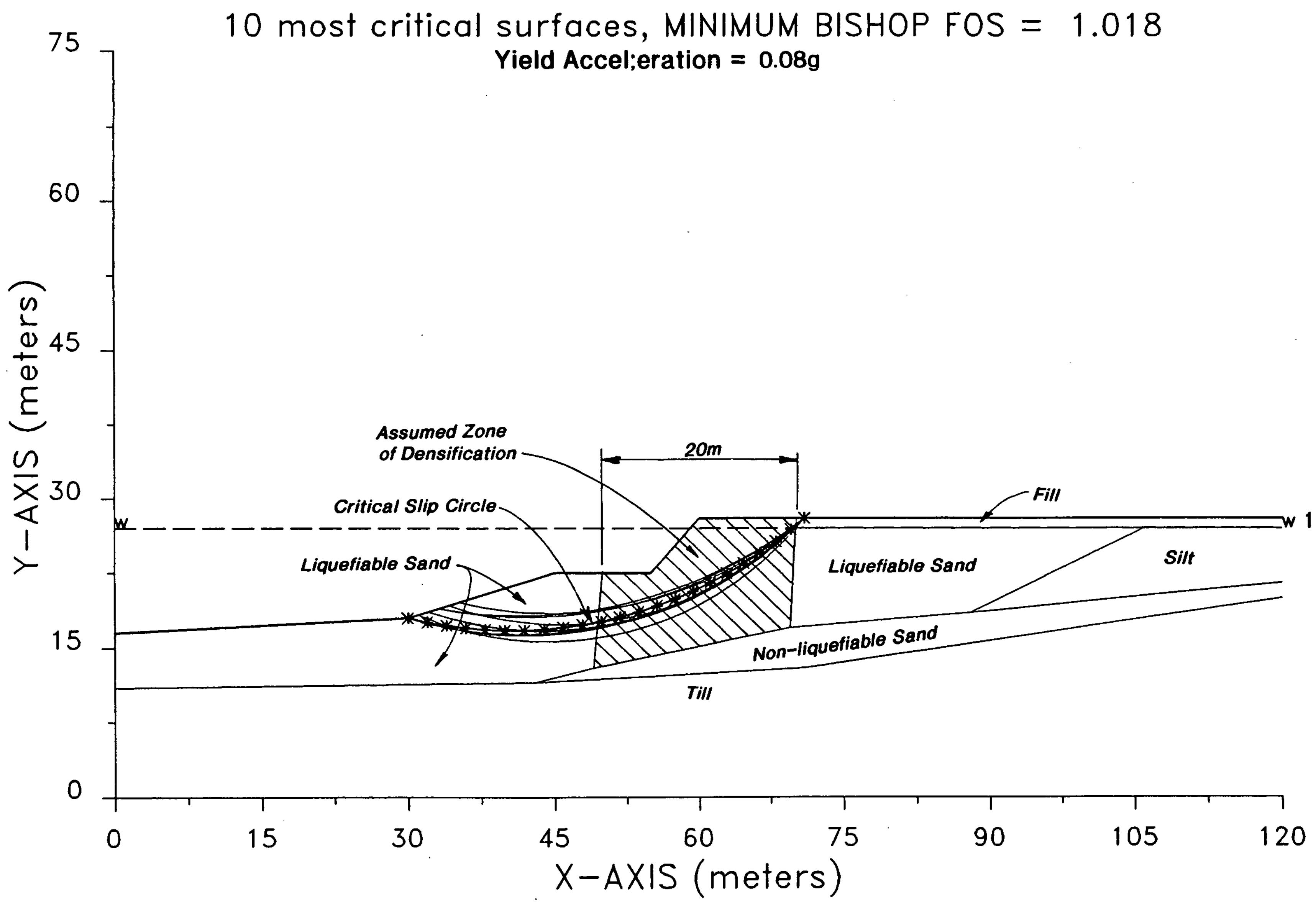


Project No. 942-1211A
Drawn w.t.
Reviewed D.W.
Date July 1995

Golder Associates

SLOPE STABILITY ANALYSIS – WITH GROUND IMPROVEMENT
CASE I – WIDTH OF DENSIFICATION BARRIER 10m
(Post-liquefaction, with Seismic Coefficient)

Figure
27



Project No. 942-1211A
Drawn w.t.
Reviewed D.W.
Date July'95

Golder Associates

SLOPE STABILITY ANALYSIS – WITH GROUND IMPROVEMENT
CASE II – WIDTH OF DENSIFICATION BARRIER 20m
(Post-liquefaction, with Seismic Coefficient)

Figure **28**

APPENDIX I

GEOTECHNICAL FIELD LOGS

PROJECT: 942-1211A

RECORD OF BOREHOLE BH 95-1

SHEET 1 OF 1

LOCATION: Fraser Gate Station, Vancouver, B.C.

BORING DATE: June 14, 1995

DATUM: G.S.

SAMPLER HAMMER,63.5kg; DROP,760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat.V - + rem.V - ⊕ Q - ● U - ○	HYDRAULIC CONDUCTIVITY, k, cm/s WATER CONTENT, PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		Ground Surface									
0	Mud Rotary - SPT Foundex	Concrete Slab		0.00							
		Compact, moist, grey-brown, silty SAND and GRAVEL, (crush) (Fill).		0.15							
1					1	DO	19				
		Very dense, cemented, grey, moist, silty SAND, some gravel (Till-like).		1.52							
2					2	DO	133				
3											
4					3	DO	102				
		End of Borehole.		4.27							
5											
6											
7											
8											
9											
10											

DATA INPUT: BAD-June'95

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: L.P.

CHECKED: D.W.

PROJECT: 942-1211A

RECORD OF BOREHOLE AH95-1

SHEET 1 OF 1

LOCATION: BC Gas/Fraser Gate/Vancouver

BORING DATE: Feb. 23, 1995

DATUM: G.S.

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat.V - + Q - ● rem.V - ⊕ U - ○	HYDRAULIC CONDUCTIVITY, k, cm/s WATER CONTENT, PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m				
		Ground Surface									
0	Solid Stem Auger Mud Bay Drilling	Compact, dry, brown, sandy GRAVEL, some silt. (FILL)	▩	0.00	1	AS					
		Compact to dense, grey, mottled, fine SAND, some gravel.	⁙	0.30	2	AS					
1 2		Dense, moist, grey, fine to medium SAND, trace gravel. (Till-Like)	⁙	0.76	3 4	AS AS					
		End of Borehole		2.29							
3											
4											
5											
6											
7											
8											
9											
10											

DATA INPUT: BAD-MAR '95

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: L.P.

CHECKED: D.W.

PROJECT: 942-1211A

RECORD OF BOREHOLE AH95-2

SHEET 1 OF 1

LOCATION: South of Railway Crossing of Pipeline

BORING DATE: Feb. 23, 1995

DATUM: G.S.

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m	HYDRAULIC CONDUCTIVITY, k, cm/s	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	20 40 60 80 SHEAR STRENGTH Cu, kPa nat.V - + Q - ● rem.V - ⊕ U - ○	WATER CONTENT, PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80		
		Ground Surface									
0	Solid Stem Auger Mud Bay Drilling	Compact, moist to wet, grey-brown, gravelly SAND to sandy GRAVEL. (FILL)	▨	0.00	1	AS					▽ Water Level in Open Hole.
1		Dense, moist, grey black, silty SAND, some gravel, trace organics. (FILL)	▨	0.46	2	AS					
2		Dense to very dense, moist, silty, fine SAND, some gravel. (Till-Like)	⁘	1.37	3	AS					
		End of Borehole		2.44							
3											
4											
5											
6											
7											
8											
9											
10											

DATA INPUT: BAD-MAR.'95

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: L.P.
CHECKED: D.W.

PROJECT: 942-1211A

RECORD OF BOREHOLE AH95-3

SHEET 1 OF 1

LOCATION: Elliot St. & Kent Ave. North Intersection.

BORING DATE: Feb. 23, 1995

DATUM: G.S.

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat.V - + Q - ● rem.V - ⊕ U - ○	HYDRAULIC CONDUCTIVITY, k, cm/s WATER CONTENT, PERCENT Wp ├──○ W──┤ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		Ground Surface									
0	Solid Stem Auger Mud Bay Drilling	0.08m Asphalt at surface. Compact to dense, dry, gravelly SAND to sandy GRAVEL. (ROAD MULCH FILL)	▨	0.00	1	AS					
1		Loose to compact, dry to moist, brown, fine to coarse SAND, trace gravel, trace organics, occasional pockets of silty SAND. (FILL)	▨	0.76	2	AS					
2					3	AS					
3					4	AS					
					5	AS					
4		Dense to very dense, grey, silty SAND, some gravel. (Till-Like)	░	3.81	6	AS					
5					7	AS					
		End of Borehole		5.33							
6											
7											
8											
9											
10											

DATA INPUT: BAD-MAR.'95

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: L.P.

CHECKED: D.W.

PROJECT: 942-1211A

RECORD OF BOREHOLE AH95-4/CPT95-1

SHEET 1 OF 1

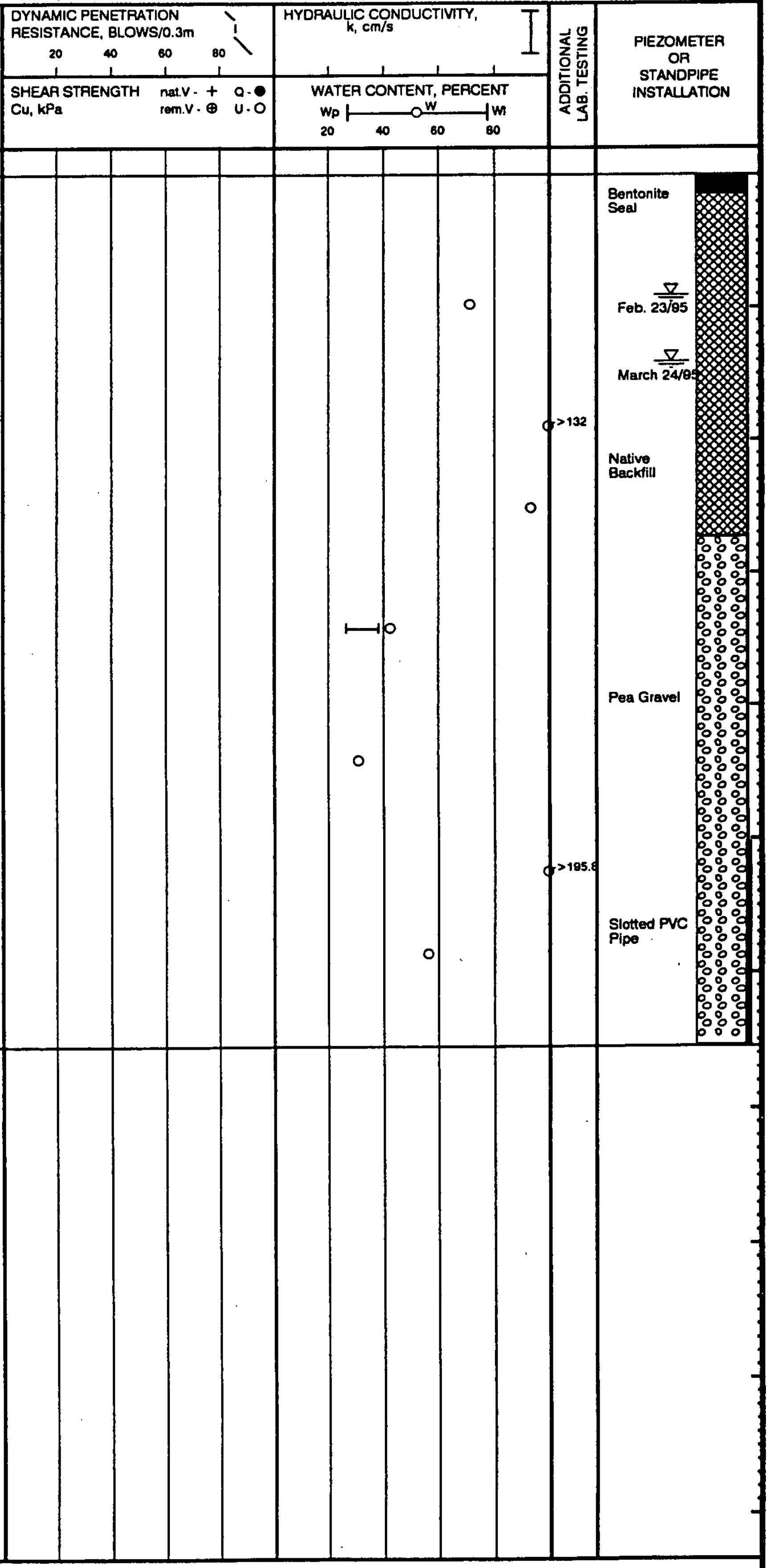
LOCATION: Fraser Gate Station Compound.

BORING DATE: Feb. 23, 1995

DATUM: G.S.

BORING METHOD: Solid Stem Auger
Mud Bay Drilling

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m
	Ground Surface				
0	Loose, moist, grey black, organic SAND. (FILL)	0.00	1	AS	
1					
	Very soft to soft, wet, brown, silty PEAT, occasional pieces of decaying wood.	1.67	2	AS	
2	Very soft to soft, moist, grey brown, organic SILT, trace sand.	2.29	3	AS	
3	Soft, moist, grey, SILT, trace to some sand, trace to some clay, trace organics (wood), interbedded lenses of fine sand.	2.74	4	AS	
4			5	AS	
5	Soft, moist, brown, fibrous to semi-fibrous PEAT, pieces of decaying wood.	5.03	6	AS	
	Soft to very soft, moist, grey, organic SILT.	5.49	7	AS	
6	Compact to dense, grey, fine to medium SAND, some organics (wood debris), trace gravel.	6.10	8	AS	
	End of Borehole	6.55			
7					
8					
9					
10					



DEPTH SCALE

1 to 50

Golder Associates

LOGGED: L.P.

CHECKED: D.W.

DATA INPUT: BAD-MAR.'95

RECORD OF BOREHOLE AH95-5/SCPT95-4

SHEET 1 OF 2

PROJECT: 942-1211A

LOCATION: Fraser Gate Station Compound.

BORING DATE: Feb. 23, 1995

DATUM: G.S.

BORING METHOD: Solid Stem Auger, Mud Bay Drilling

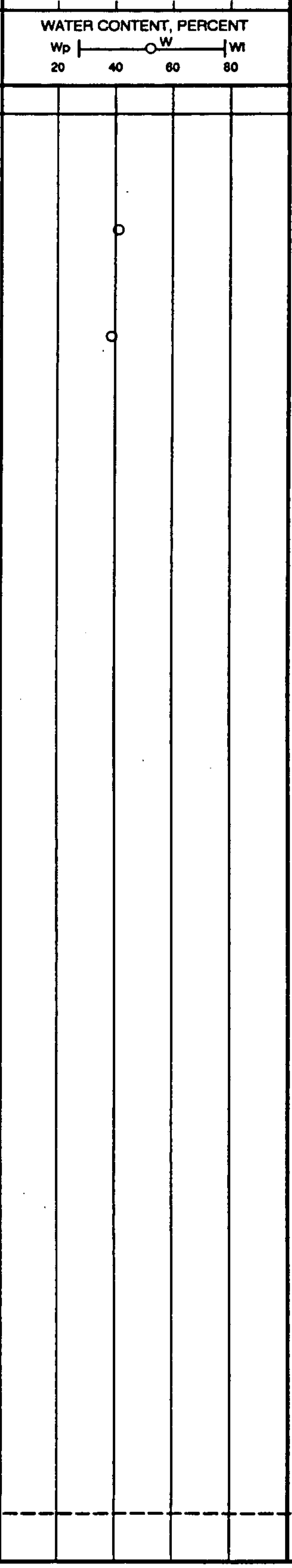
DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m
0	Ground Surface				
0–2	Loose to compact, moist, brown black SAND, some organic, trace to some gravel, trace silt. (FILL)	0.00	1 2	AS AS	
2–3.5	Loose, wet, grey to brown, fine to coarse SAND, trace organics.	1.98	3 4	AS AS	
3.5–6.1	Loose, wet, grey, silty, fine SAND to sandy SILT, trace to some organics.	3.50	5 6 7 8	AS AS AS AS	
6.1–10	Loose to compact, wet, grey, fine SAND, trace to some silt, interbedded lenses of sandy silt.	6.10	9 10 11 12 13	AS AS AS AS AS	
10	CONTINUED ON NEXT PAGE				

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20, 40, 60, 80

SHEAR STRENGTH Cu, kPa: nat.V - + Q - ● rem.V - ⊕ U - O

HYDRAULIC CONDUCTIVITY, k, cm/s

WATER CONTENT, PERCENT



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

Water Level in Open Hole.

DATA INPUT: BAD-MAR.'95

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: L.P.

CHECKED: D.W.

PROJECT: 942-1211A

RECORD OF BOREHOLE AH95-5/SCPT95-4

SHEET 2 OF 2

LOCATION: Fraser Gate Station Compound.

BORING DATE: Feb. 23, 1995

DATUM: G.S.

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat.V - + Q - ● rem.V - ⊕ U - O	HYDRAULIC CONDUCTIVITY, k, cm/s WATER CONTENT, PERCENT Wp ├─O W─┤ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		CONTINUED FROM PREVIOUS PAGE									
10	Solid Stem Auger Mud Bay Drilling	As above.			13	AS					
		Compact to dense, grey, silty, fine to coarse SAND, some gravel.		10.30							
11					14	AS					
					15	AS					
12											
		End of Borehole		12.20							
13											
14											
15											
16											
17											
18											
19											
20											

DATA INPUT: BAD-MAR.'95

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: L.P.

CHECKED: D.W.

PROJECT: 942-1211A

RECORD OF BOREHOLE AH95-6/CPT95-3

SHEET 1 OF 2

LOCATION: Fraser Gate Station Compound.

BORING DATE: Feb. 23, 1995

DATUM: G.S.

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat.V - + Q - ● rem.V - ⊕ U - ○

HYDRAULIC CONDUCTIVITY, k, cm/s

WATER CONTENT, PERCENT Wp ⊢—○W—⊣ Wl: 20 40 60 80

ADDITIONAL LAB. TESTING

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	ELEV. DEPTH (m)	NUMBER	TYPE	PIEZOMETER OR STANDPIPE INSTALLATION
		Ground Surface				
0	Solid Stem Auger Mud Bay Drilling	Loose to compact, grey, moist, silty SAND, some organics. (FILL)	0.00			Bentonite Seal
		Loose, moist, grey, silty SAND to sandy SILT, some organics (wood fragments). (FILL)	0.46			Native Backfill
				1	AS	
1						
2				2	AS	
						Feb. 23/95
		Loose to compact, wet, grey, fine SAND, some silt to silty fine SAND with occasional lenses of silt.	2.74			March 24/95
3				3	AS	Sand Sloughed
4		Loose to compact, wet, grey, fine SAND, trace silt.	4.27			Slotted PVC Pipe
5				4	AS	
6						
				5	AS	
7						
		Loose to compact, grey, wet, fine to coarse SAND, trace silt.	7.77			
8				6	AS	
9						
				7	AS	
10		CONTINUED ON NEXT PAGE				

DATA INPUT: BAD-MAR.'95

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: L.P.
CHECKED: D.W.

PROJECT: 942-1211A

RECORD OF BOREHOLE AH95-6/CPT95-3

SHEET 2 OF 2

LOCATION: Fraser Gate Station Compound.
BORING DATE: Feb. 23, 1995
DATUM: G.S.

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat.V - + Q - ● rem.V - ⊕ U - ○	HYDRAULIC CONDUCTIVITY, k, cm/s WATER CONTENT, PERCENT Wp ├──○W──┤ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m				
10	Solid Stem Auger Mud Bay Drilling	CONTINUED FROM PREVIOUS PAGE									
		As above.									
11		Compact to dense, wet, grey, fine to coarse SAND, trace gravel.		11.00	8	AS					
12					9	AS					
13											
14		Dense, moist, grey, silty SAND, some gravel. (Till-Like)		14.02	10	AS					
15		End of Borehole		15.24							
16											
17											
18											
19											
20											

DATA INPUT: BAD-MAR.'95

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: L.P.

CHECKED: D.W.

PROJECT: 942-1211A

RECORD OF BOREHOLE AH95-7/CPT 95-5

SHEET 1 OF 1

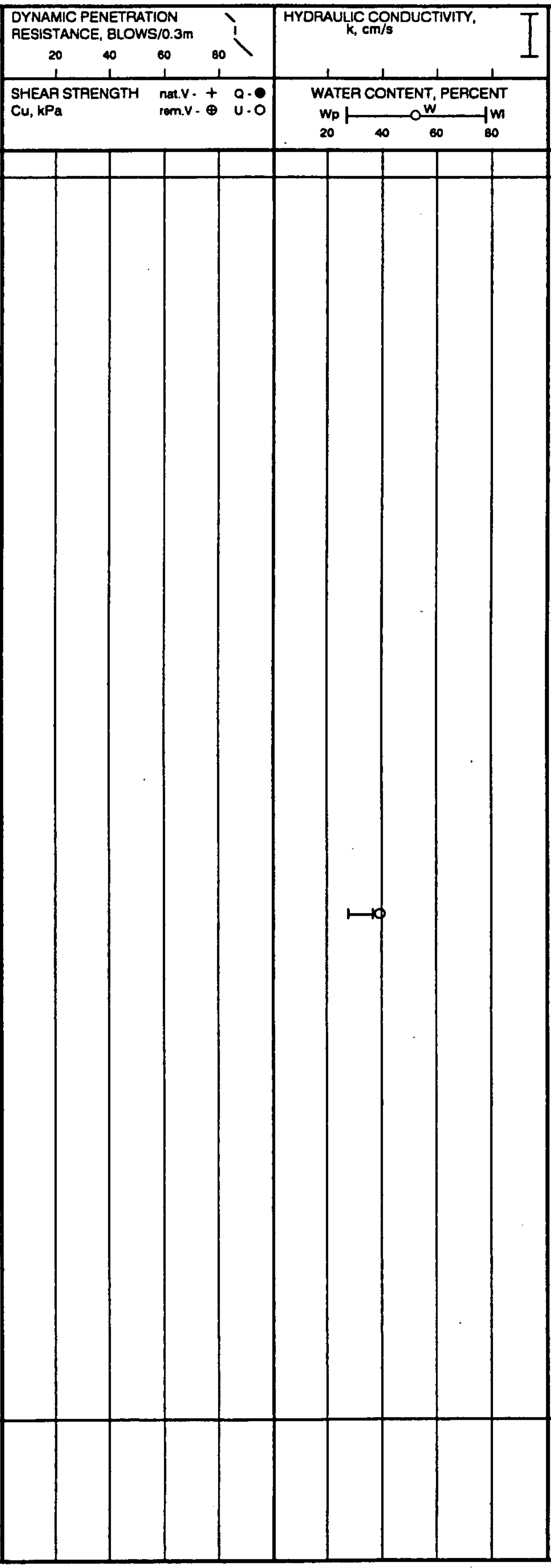
LOCATION: Fraser Gate Station Compound.

BORING DATE: March 24,1995

DATUM: G.S.

BORING METHOD: Solid Stem Auger / Mud Bay Drilling

DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m
Ground Surface				
Loose, grey SAND and SILT, some wood pieces. (FILL).	0.00			
Loose, moist, grey, organic, sandy SILT, some decaying wood pieces.	0.30	1	AS	
Very soft to soft, moist to wet, grey SILT, trace fine sand, trace clay, trace organics, some fine sand to silt and fine to medium sand below 7.0m	1.67	2 3 4 5	AS AS AS AS	
Dense, fine to medium SAND, some cobbles, trace silt.	7.77	6	AS	
Very dense SAND and GRAVEL, some cobbles.	8.83	7	AS	
End of Borehole.	9.29			



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

Water Level in Open Hole, March 24/95.

DEPTH SCALE METRES: 0 1 2 3 4 5 6 7 8 9 10

DATA INPUT: BAD-MAR.'95

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: T.S.

CHECKED: D.W.

RECORD OF BOREHOLE AH95-8

PROJECT: 942-1211A
LOCATION: BC Gas/Fraser Gate/Vancouver
BORING DATE: March 24,1995
SHEET 1 OF 1
DATUM: G.S.

BORING METHOD: MUD BAY TRUCK MOUNTED AUGER / SOLID STEM AUGER

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80
SHEAR STRENGTH Cu, kPa: nat.V - + Q - ● rem.V - ⊕ U - ○
HYDRAULIC CONDUCTIVITY, k, cm/s
WATER CONTENT, PERCENT: Wp ⊢—○W—⊣ Wl 20 40 60 80

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Ground Surface							
	Loose, grey SAND and SILT, some wood pieces. (FILL).	▩	0.00					
1	Soft, moist, grey, organic, sandy SILT, some decaying wood pieces.	⁚	0.30	1	AS			
2	Soft, moist, grey SILT, trace fine sand, trace clay, trace organics.	‖	1.67	2	AS			
3								▽
4				3	AS			
5				4	AS			
6								
7	Loose, wet, grey SILT, some fine sand to silt and fine to medium SAND.	‖	6.85	5	AS			
8	Dense, fine to medium SAND, some cobbles, trace silt.	⁚	7.77	6	AS			
9	Very dense SAND and GRAVEL, some cobbles.	⁂	8.83	7	AS			
	End of Borehole.		9.29					
10								

DATA INPUT: BAD-MAR-'95

DEPTH SCALE
1 to 50

Golder Associates

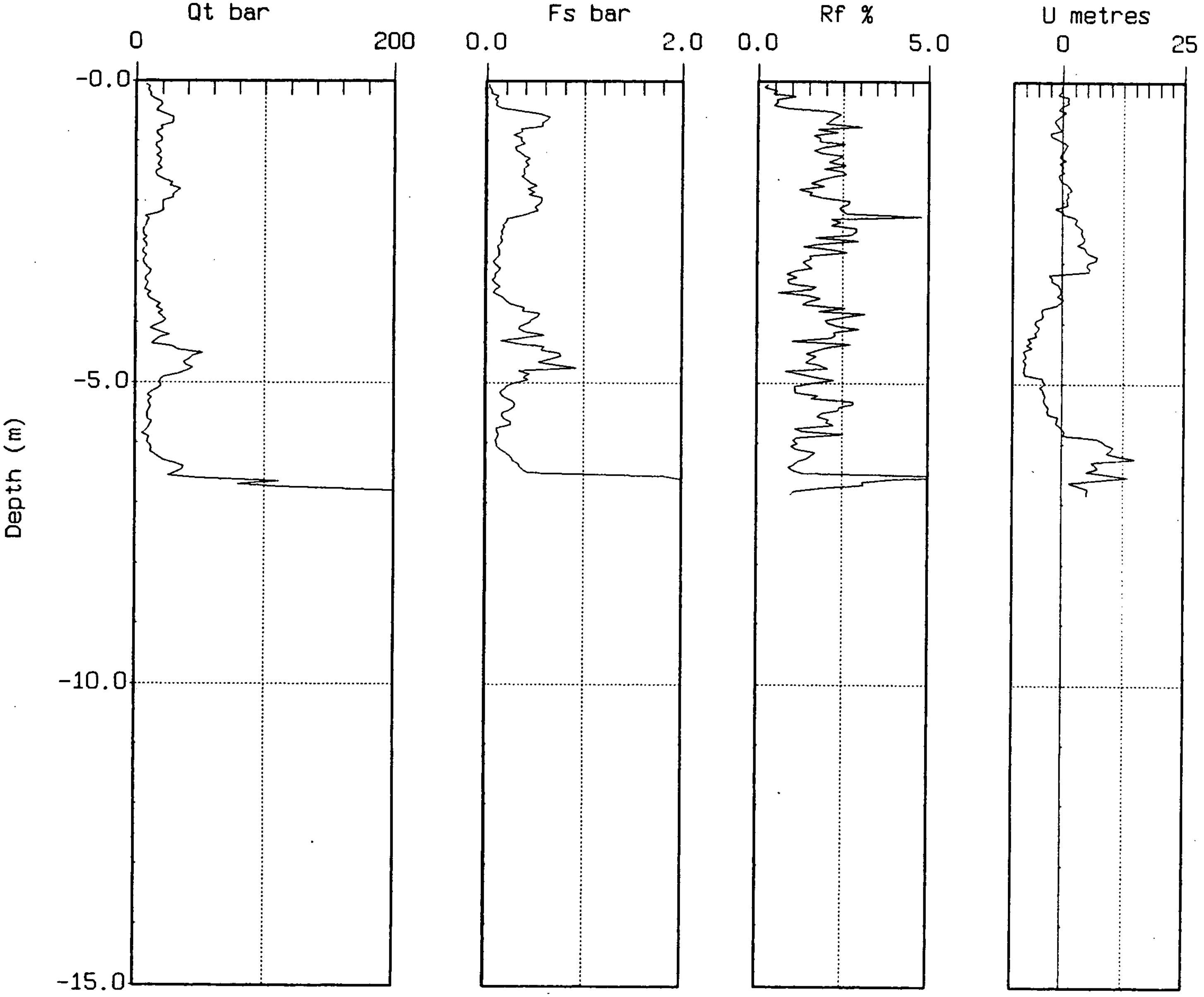
LOGGED: L.P.
CHECKED: D.W.

CONETEC

GOLDER ASSOCIATES

CPT 95-1/AH 95-4
Fraser Gate Station

Cone: 10 TON AD 028
Date: 02:23:95 12:27



Max. Depth: 6.85 (m)

Depth Inc.: 0.05 (m)

INFERRED SOIL STRATIGRAPHY

	Loose organic SAND, (fill).
1.7	*Soft PEAT*
2.3	*Very soft to soft organic SILT.*
2.7	*Soft SILT, trace to some sand, trace to some clay.*
5.0	*Soft PEAT.*
5.5	*Soft organic SILT.*
6.1	*Compact to dense SAND.*
6.9	*End of Hole (refusal)*

NOTE:

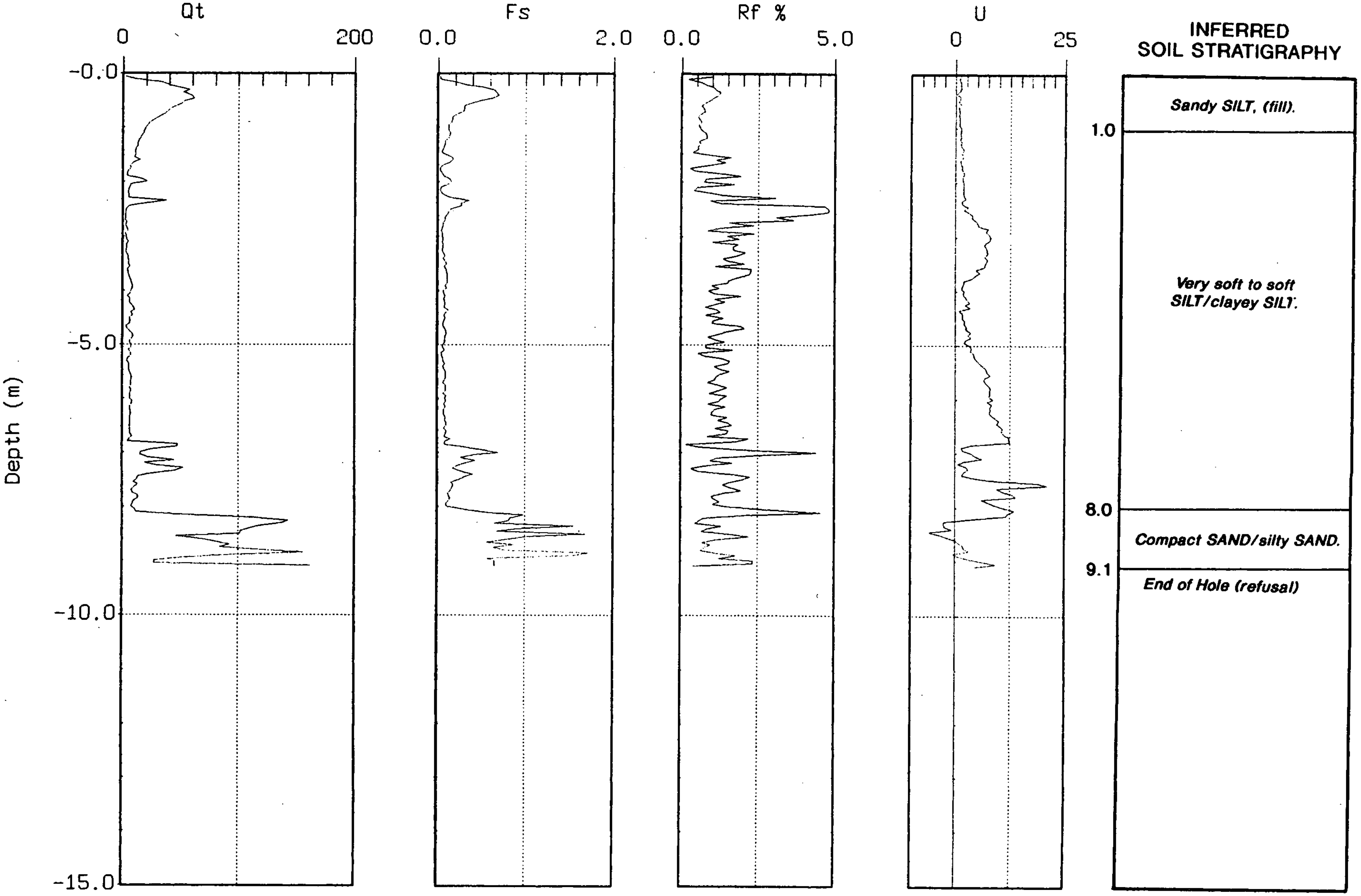
Soil stratigraphy based on augerhole AH 95-4 data at same location.

CONETEC

GOLDER ASSOCIATES

CPT 95-2
Fraser Gate Station

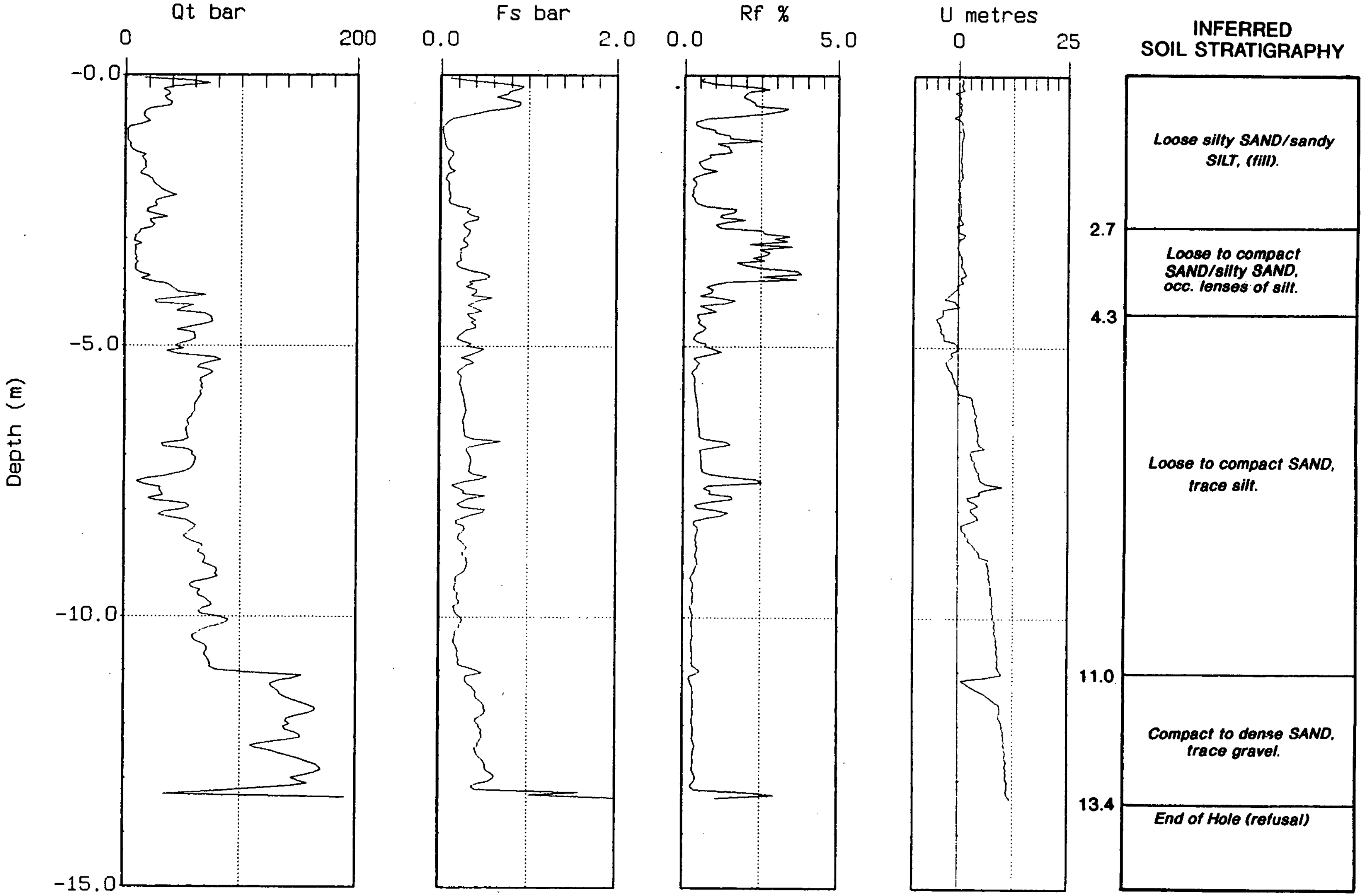
Cone: 10 TON AD 028
Date: 02:23:95 13:35



Max. Depth: 9.10 (m)
Depth Inc.: 0.05 (m)

GOLDER ASSOCIATES **CPT 95-3/AH 95-6**
Fraser Gate Station

Cone: 10 TON AD 028
Date: 02:23:95 14:08



Max. Depth: 13.35 (m)

Depth Inc.: 0.05 (m)

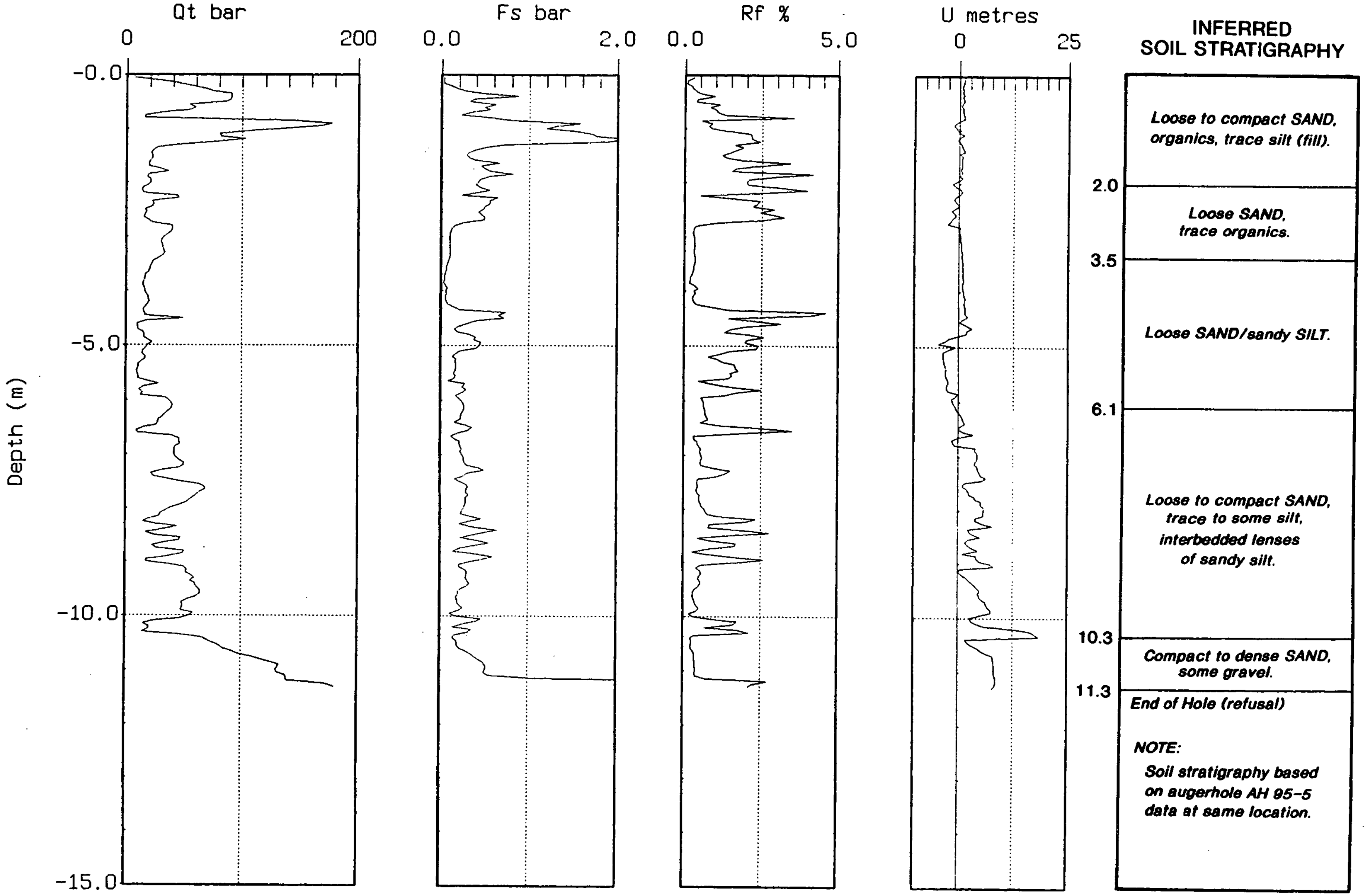
NOTE:
Soil stratigraphy based on augerhole AH 95-6 data at same location.

CONETEC

GOLDER ASSOCIATES

SCPT 95-4/AH 95-5
Fraser Gate Station

Cone: 10 TON AD 028
Date: 02:23:95 15:21



Max. Depth: 11.30 (m)

Depth Inc.: 0.05 (m)

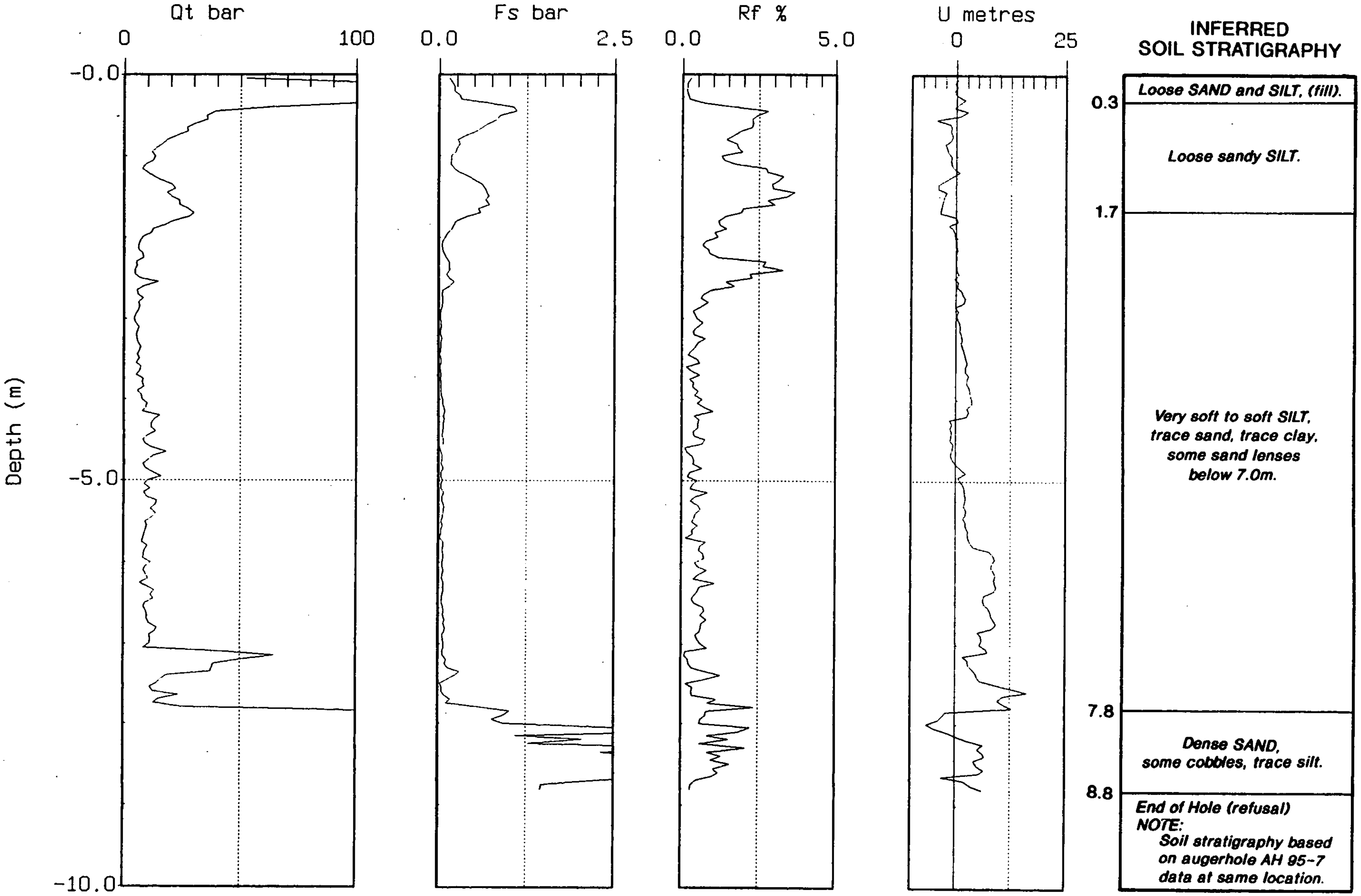
CONETEC

Golder

CPT 95-5/AH 95-7

Fraser Gate Station

Cone: 10 TON A 027
Date: 03:24:95 09:20



Max. Depth: 8.80 (m)

Depth Inc.: 0.05 (m)

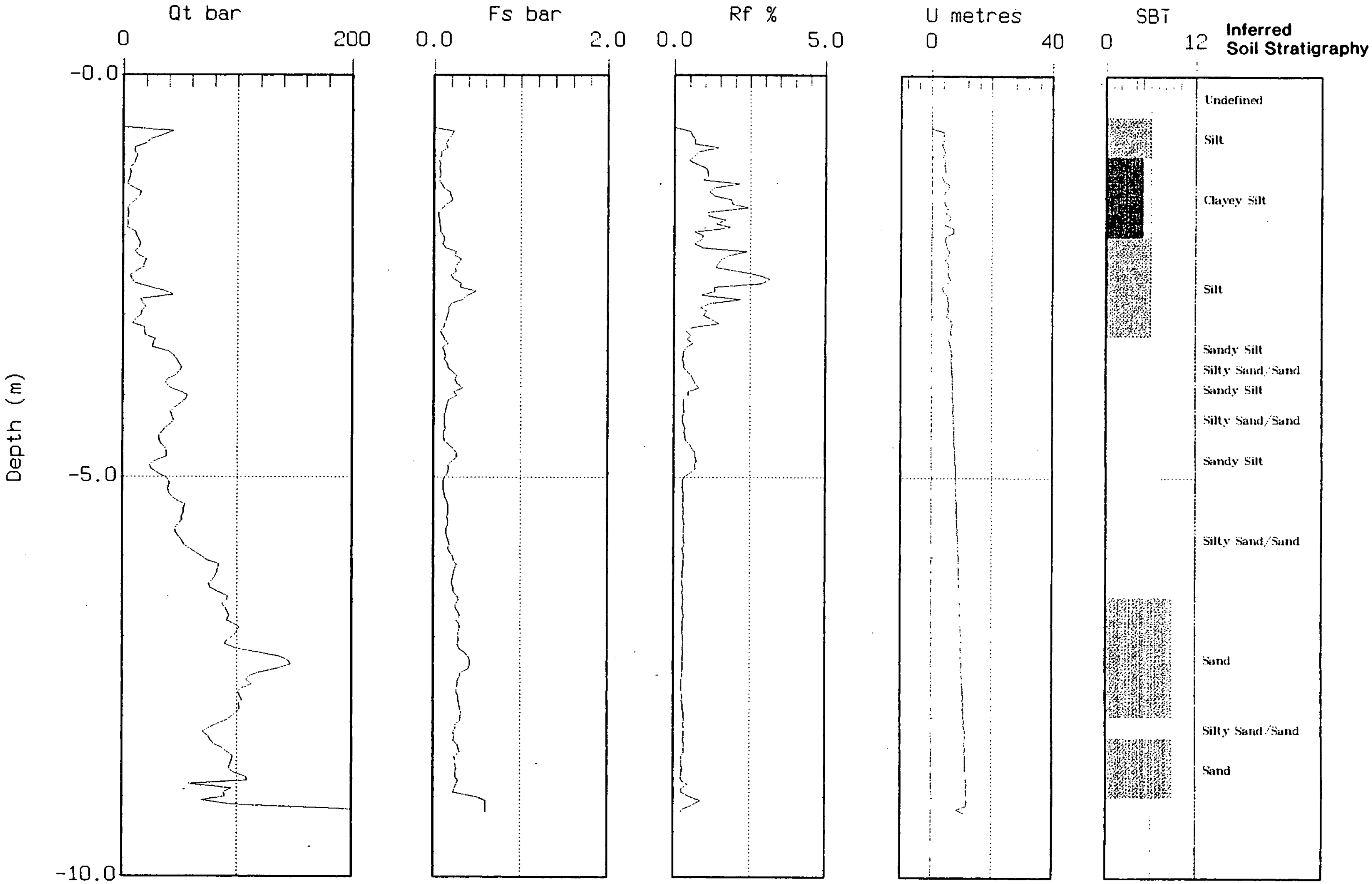
CONETEC

Golder Associates

CPT 95-6

Offshore – South of Fraser Gate Station

Cone: 10 TON A 035
Date: 04:03:95 08:17



Max. Depth: 9.20 (m)

Depth Inc.: 0.05 (m)

SBT: Soil Behavior Type (Robertson and Campanella 1988)

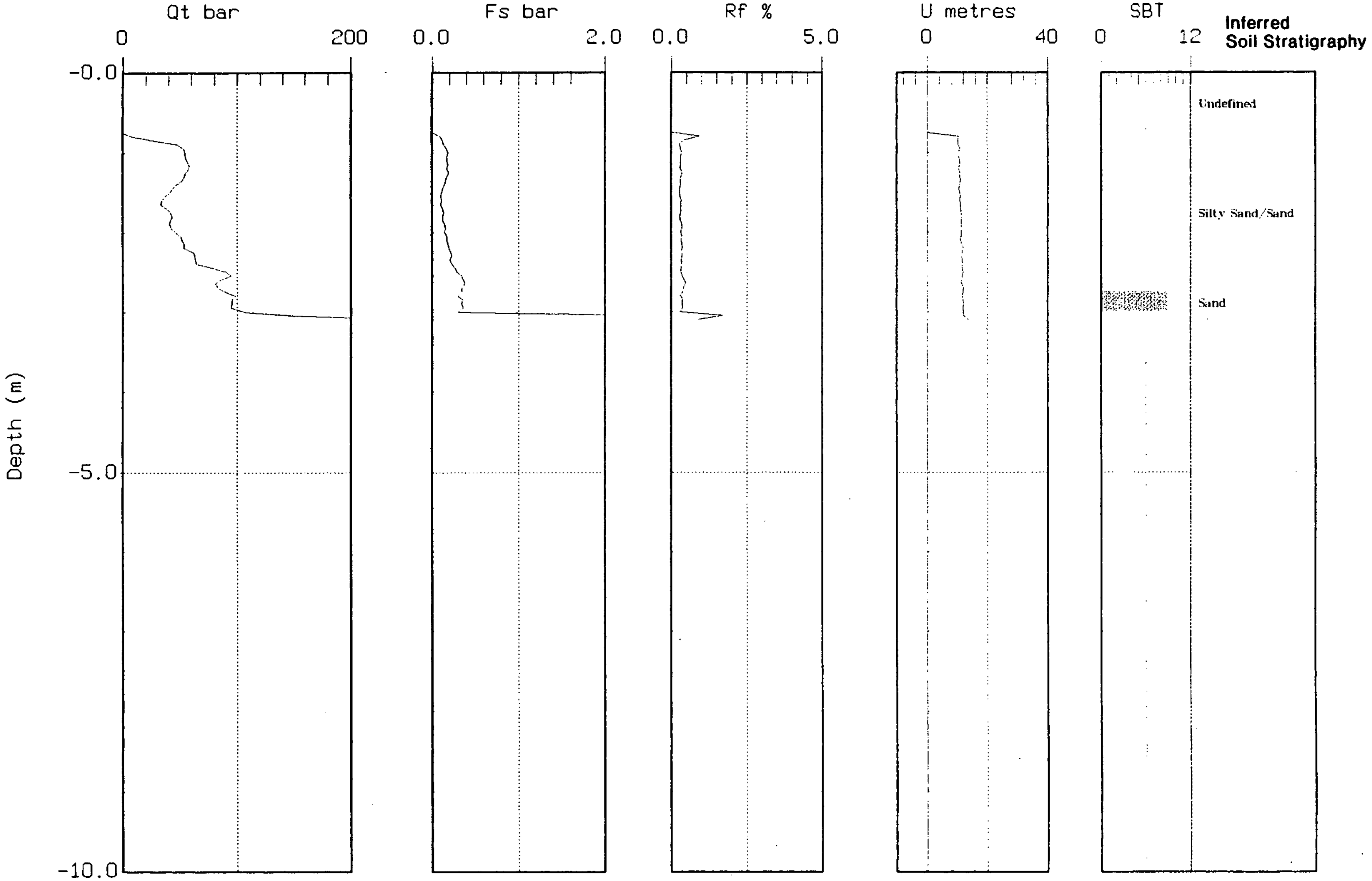
CONETEC

Golder Associates

CPT 95-7

Offshore – South of Fraser Gate Station

Cone: 10 TON A 308
Date: 04:08:95 10:04



Max. Depth: 3.10 (m)

Depth Inc.: 0.05 (m)

SBT: Soil Behavior Type (Robertson and Campanella 1988)

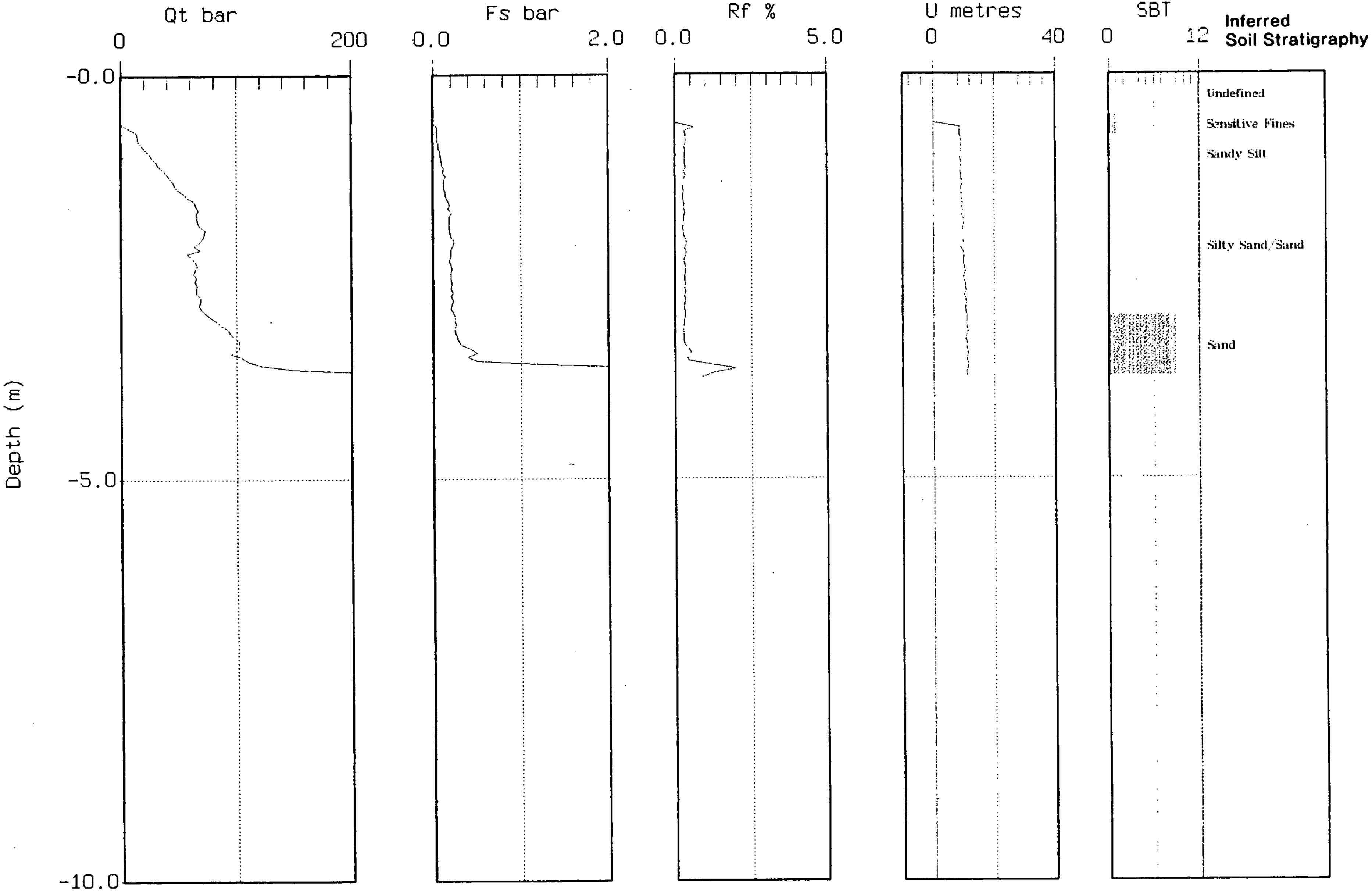
CONETEC

Golder Associates

CPT 95-8

Offshore – South of Fraser Gate Station

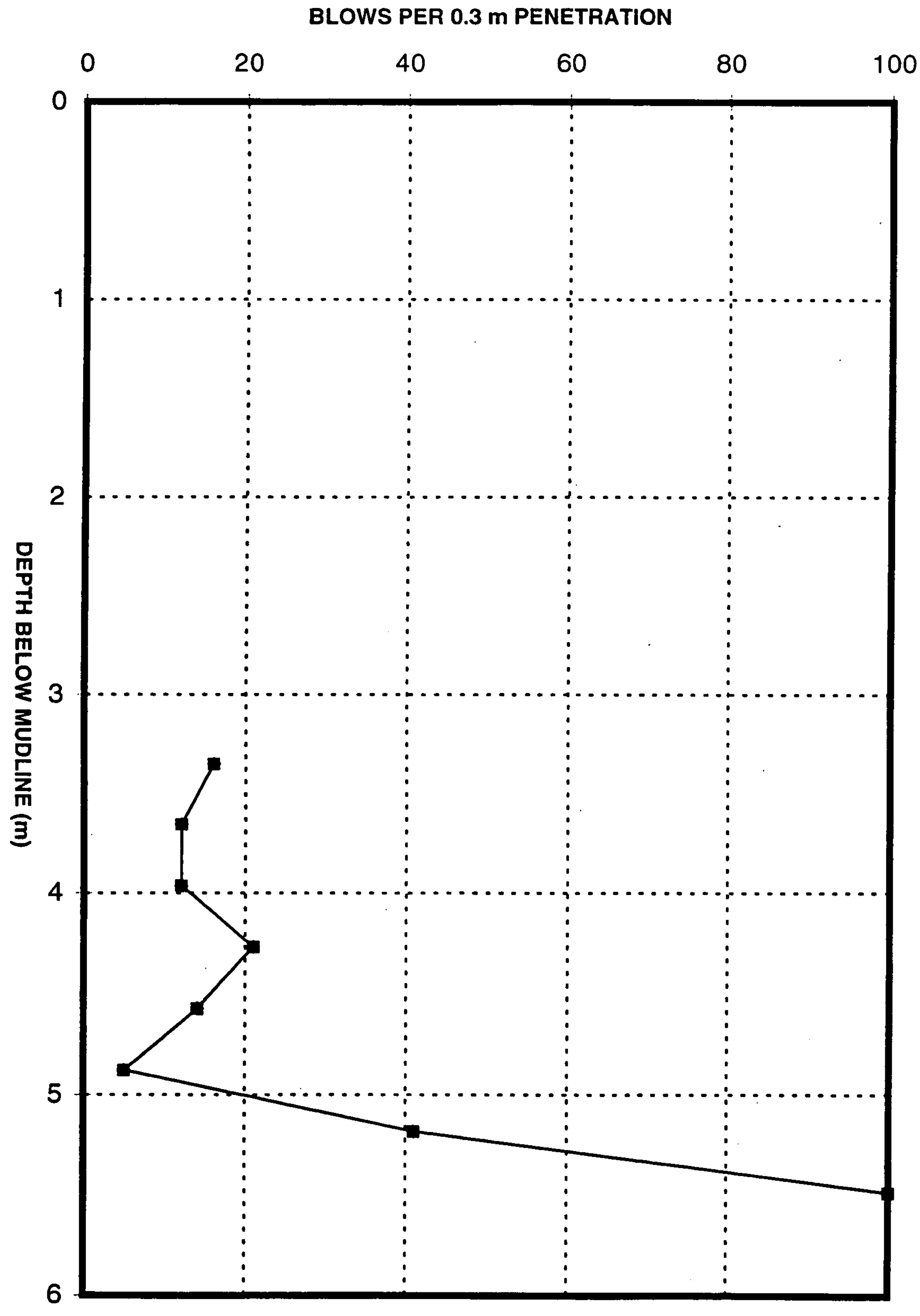
Cone: 10 TON A 009
Date: 04:02:96 11:16



Max. Depth: 3.75 (m)

Depth Inc.: 0.05 (m)

SBT: Soil Behavior Type (Robertson and Campanella 1988)



Weight of hammer = 63.5 kg. Drop = 760mm

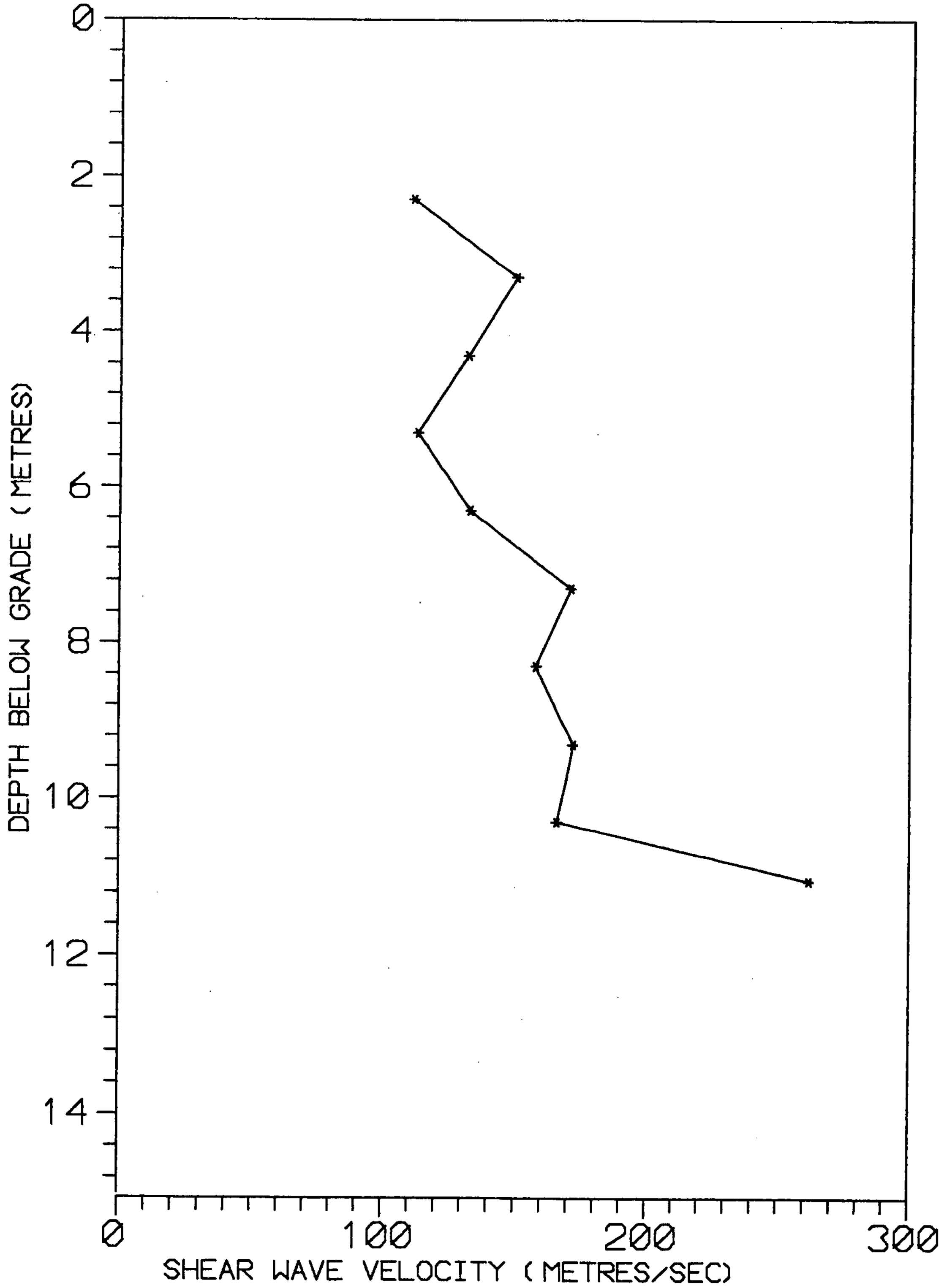
Project No. ....*942–1211A*.... Drawn ....*w.l.*.... Reviewed ....*D.W.*.... Date ....*July'95*....

Golder Associates

RESULTS OF DYNAMIC CONE PENETRATION TEST AT CPT 95–8

Figure

I–a



Project No. .... Drawn .... Reviewed .... Date ....

Golder Associates

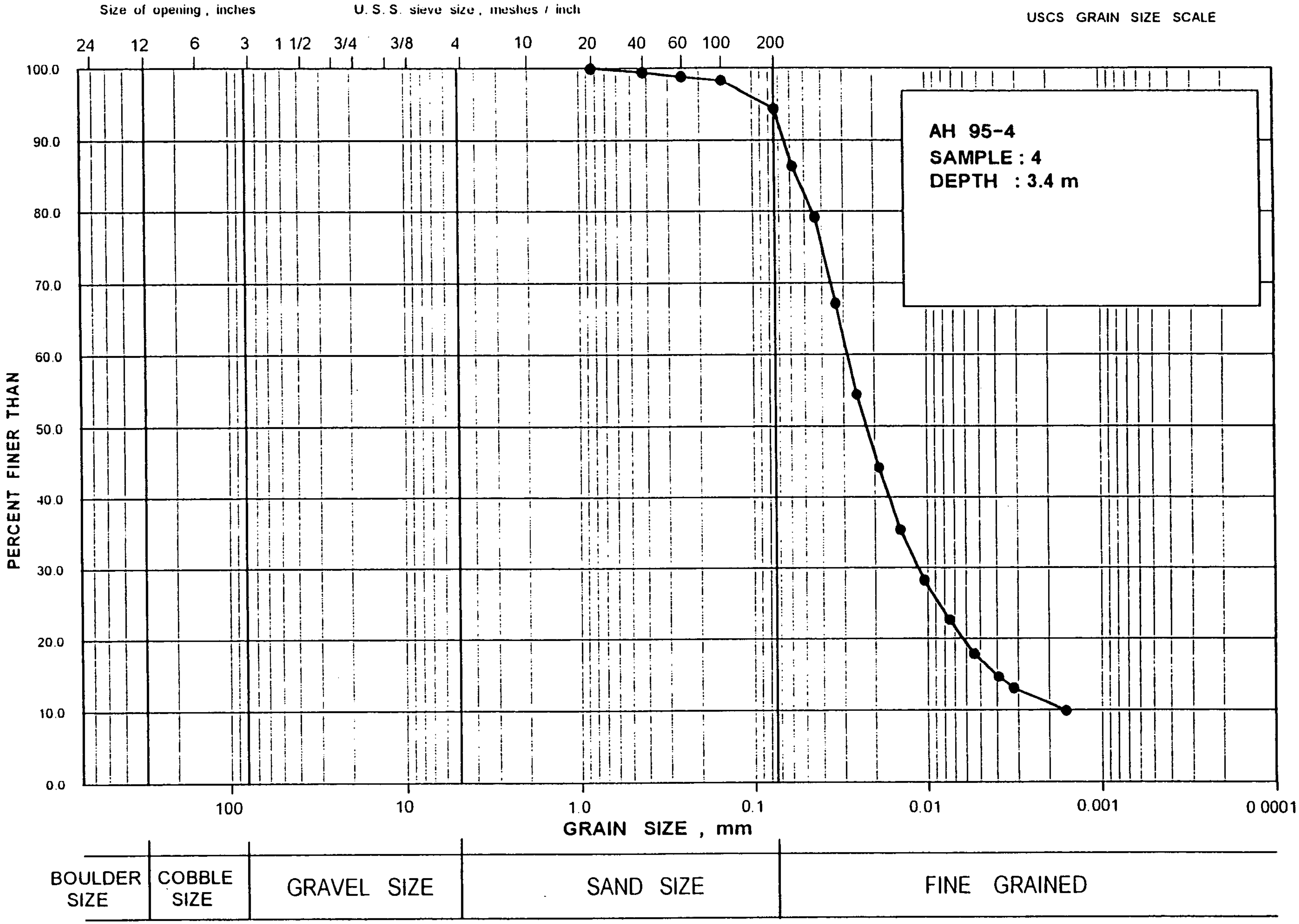
MEASURED DOWN-HOLE SHEAR WAVE VELOCITY Vs DEPTH AT TEST HOLE SCPT 95-4

Figure

I-b

APPENDIX II

RESULTS OF GRADATION TESTING



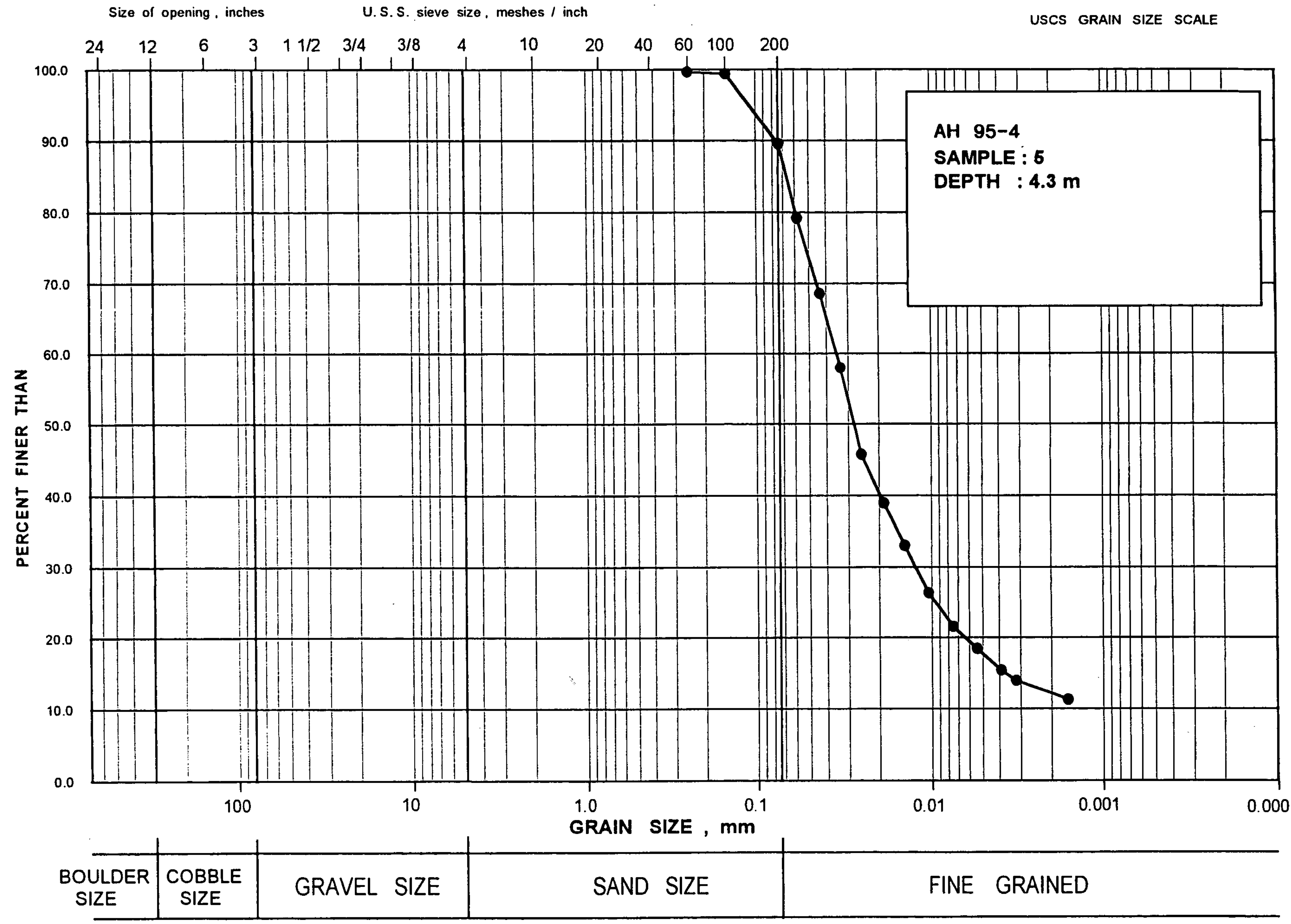
Project No. ..9421211A...
Drawn ...............M.O.......
Reviewed ....
Date ....03/14/95.....

Golder Associates

GRAIN SIZE DISTRIBUTION

Figure

II–a



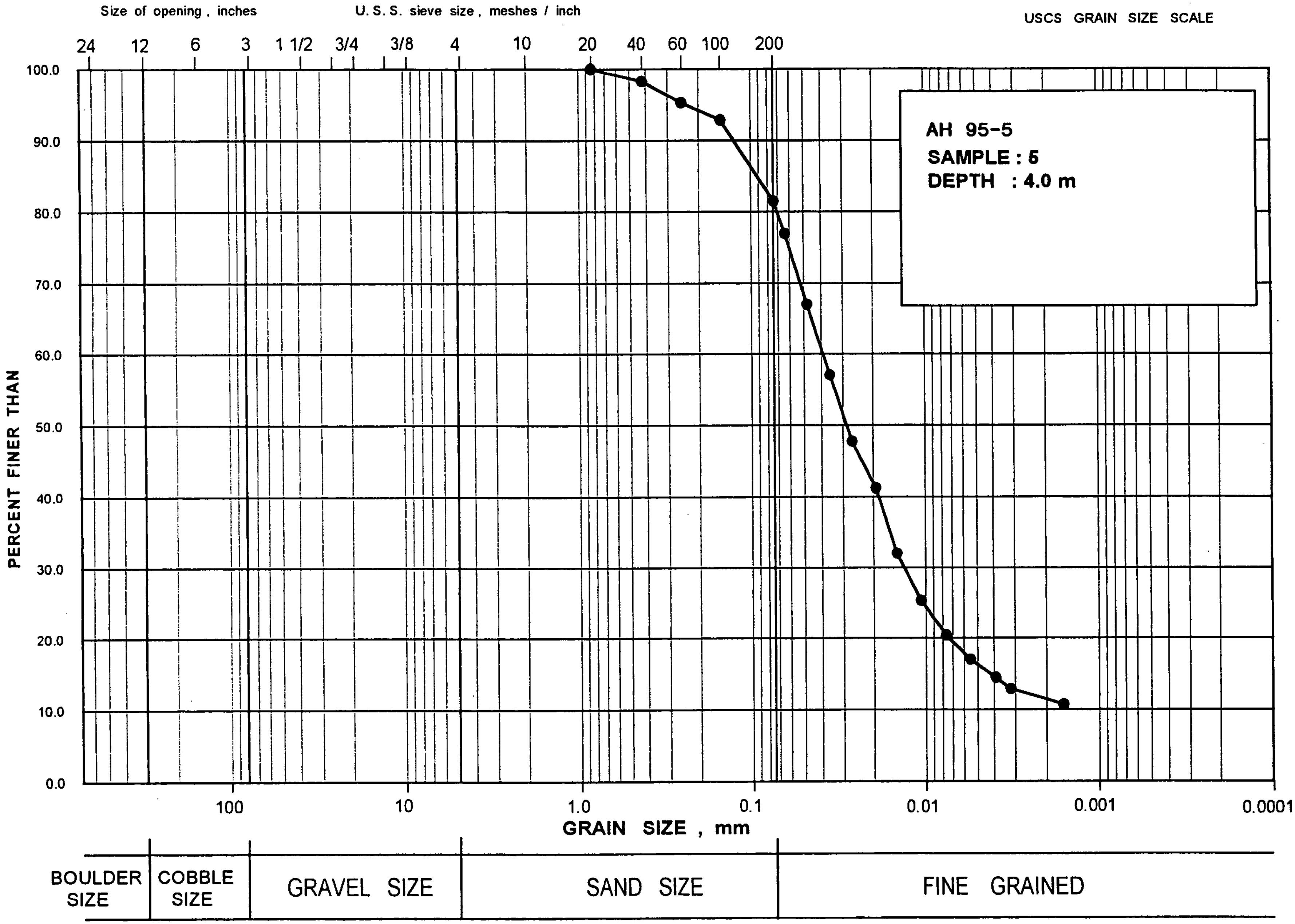
Project No. ..942121.1A....
Drawn ................M.O.......
Reviewed ....
Date ....03/14/95......

Golder Associates

GRAIN SIZE DISTRIBUTION

Figure

II–b



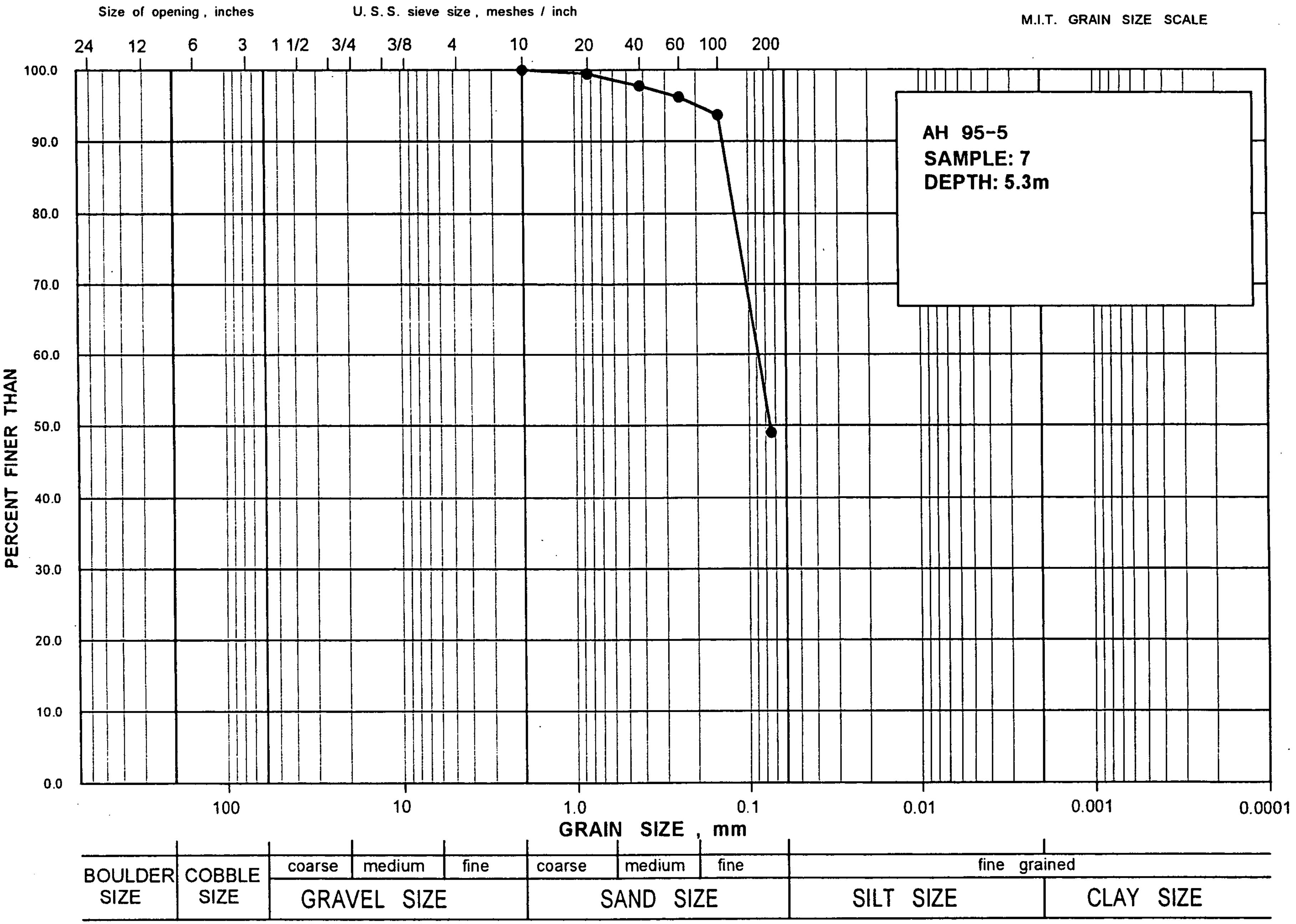
Project No. ..9421211A...
Drawn ...............M.O........
Reviewed ....
Date ....03/15/95......

Golder Associates

GRAIN SIZE DISTRIBUTION

Figure

II-C



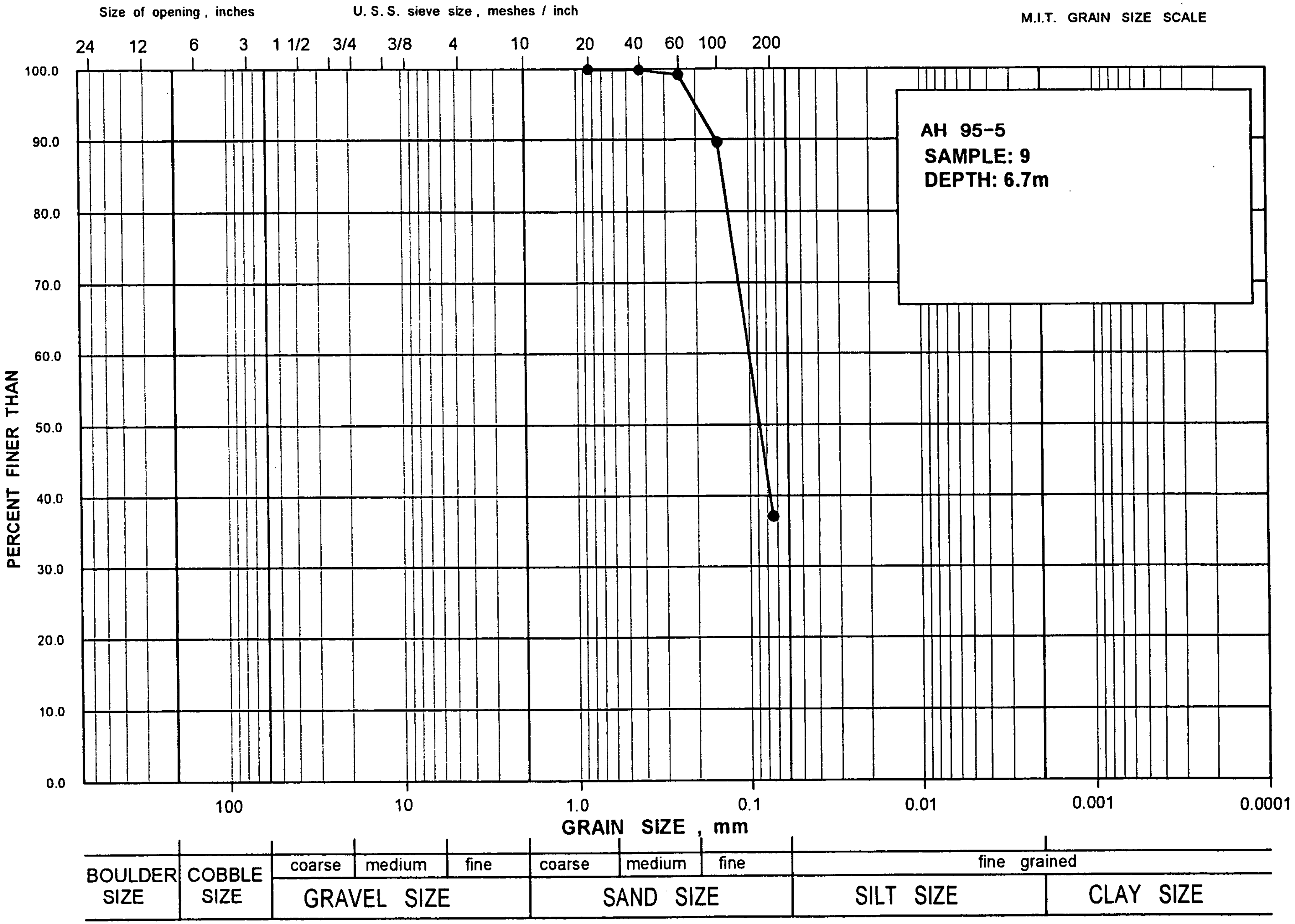
Project No. ..9421211A....
Drawn ....MP...............
Reviewed ....
Date ....03/13/95.....

Golder Associates

GRAIN SIZE DISTRIBUTION

Figure

II-d



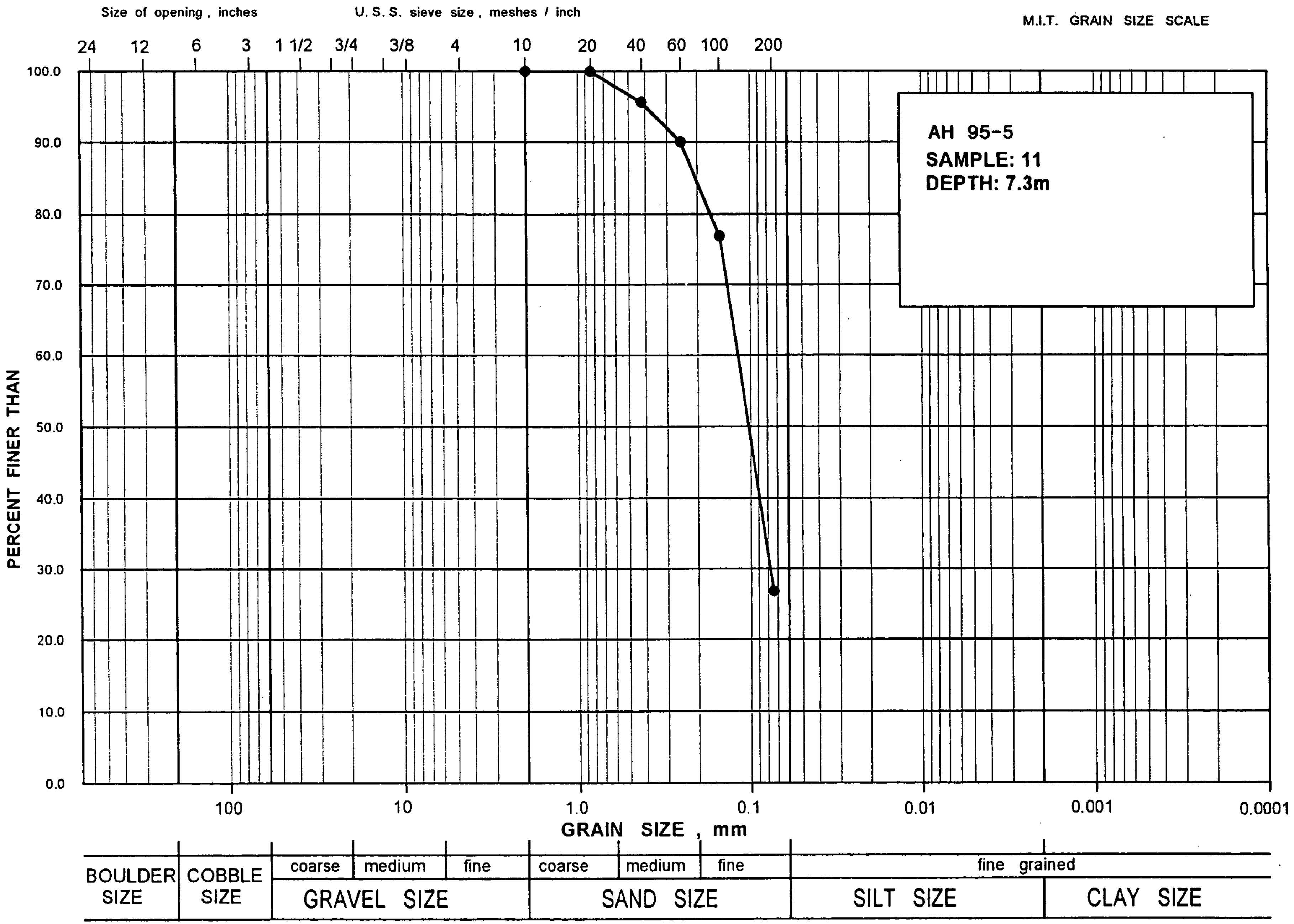
Project No. ..9421211A....
Drawn ....MP...............
Reviewed ....
Date ....03/13/95......

Golder Associates

GRAIN SIZE DISTRIBUTION

Figure

II–e



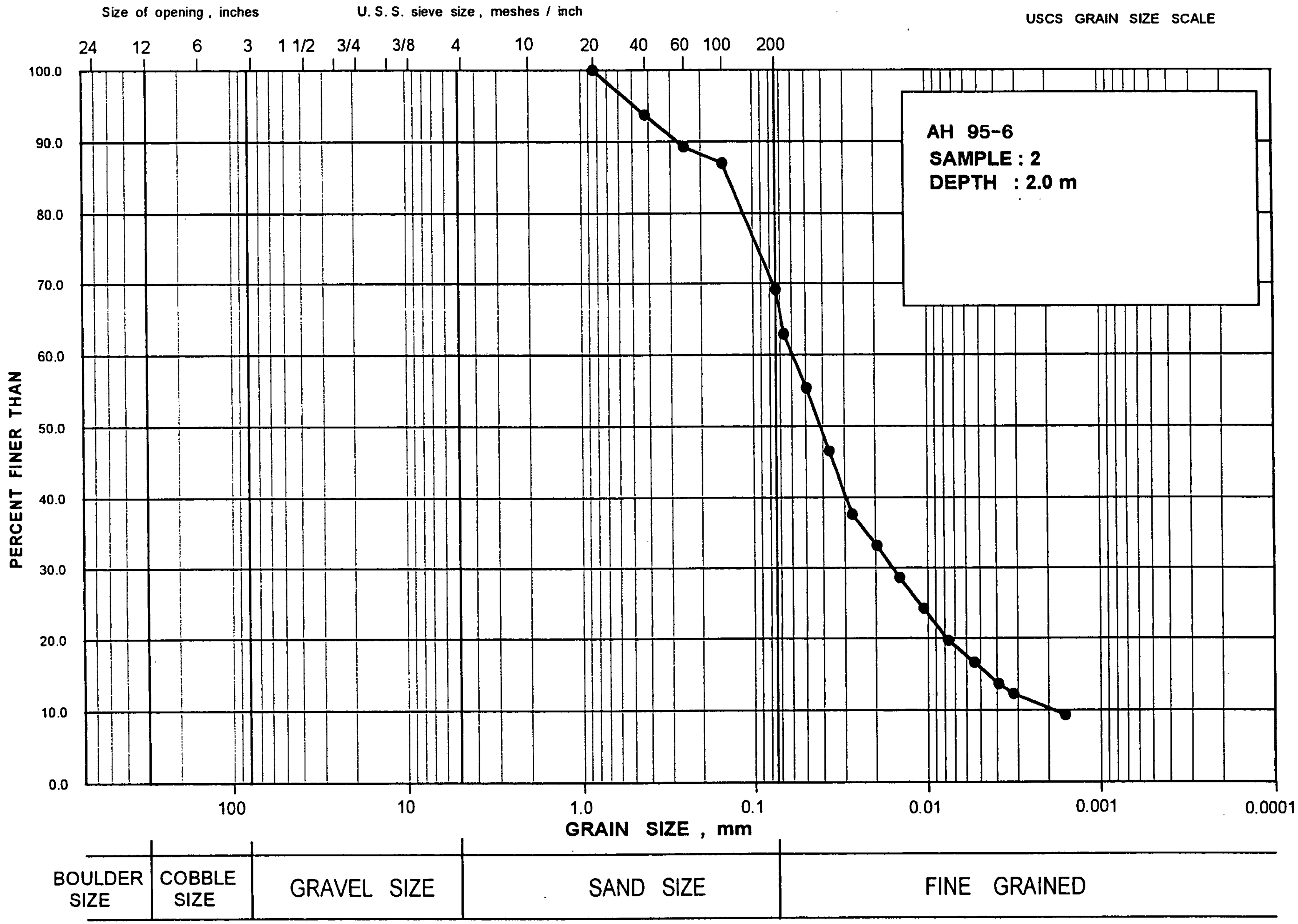
Project No. ..9421211A...
Drawn ....MP...............
Reviewed ....
Date ....03/13/95......

Golder Associates

GRAIN SIZE DISTRIBUTION

Figure

II–f



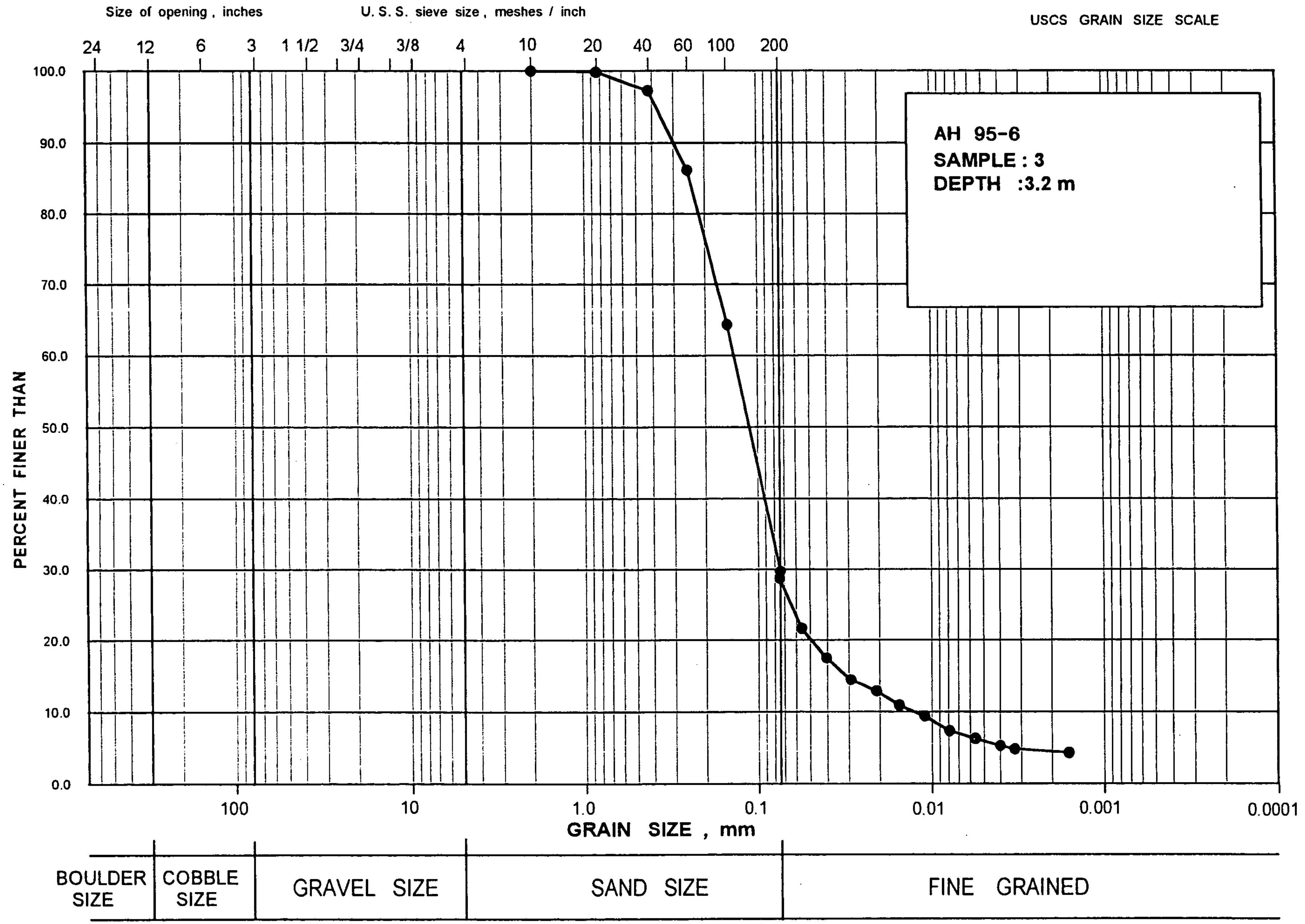
Project No. ..9421211A....
Drawn ...............M.O.......
Reviewed ....
Date ....03/15/95......

Golder Associates

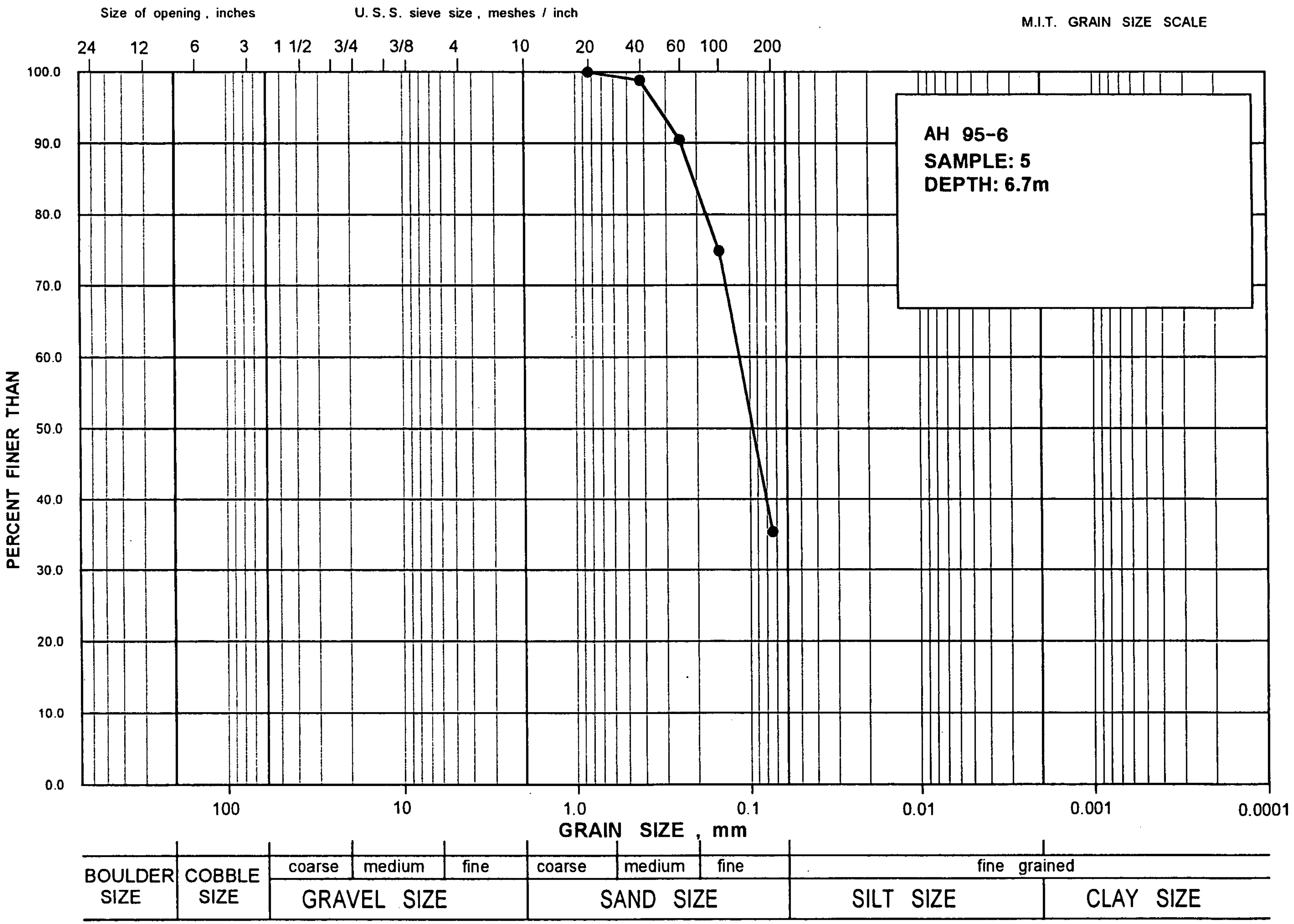
GRAIN SIZE DISTRIBUTION

Figure

II–g



Project No. ..9421211A.... **Drawn** ................M.O........ **Reviewed** .... **Date** ....03/15/95......	Golder Associates	**GRAIN SIZE DISTRIBUTION**	**Figure** **II–h**



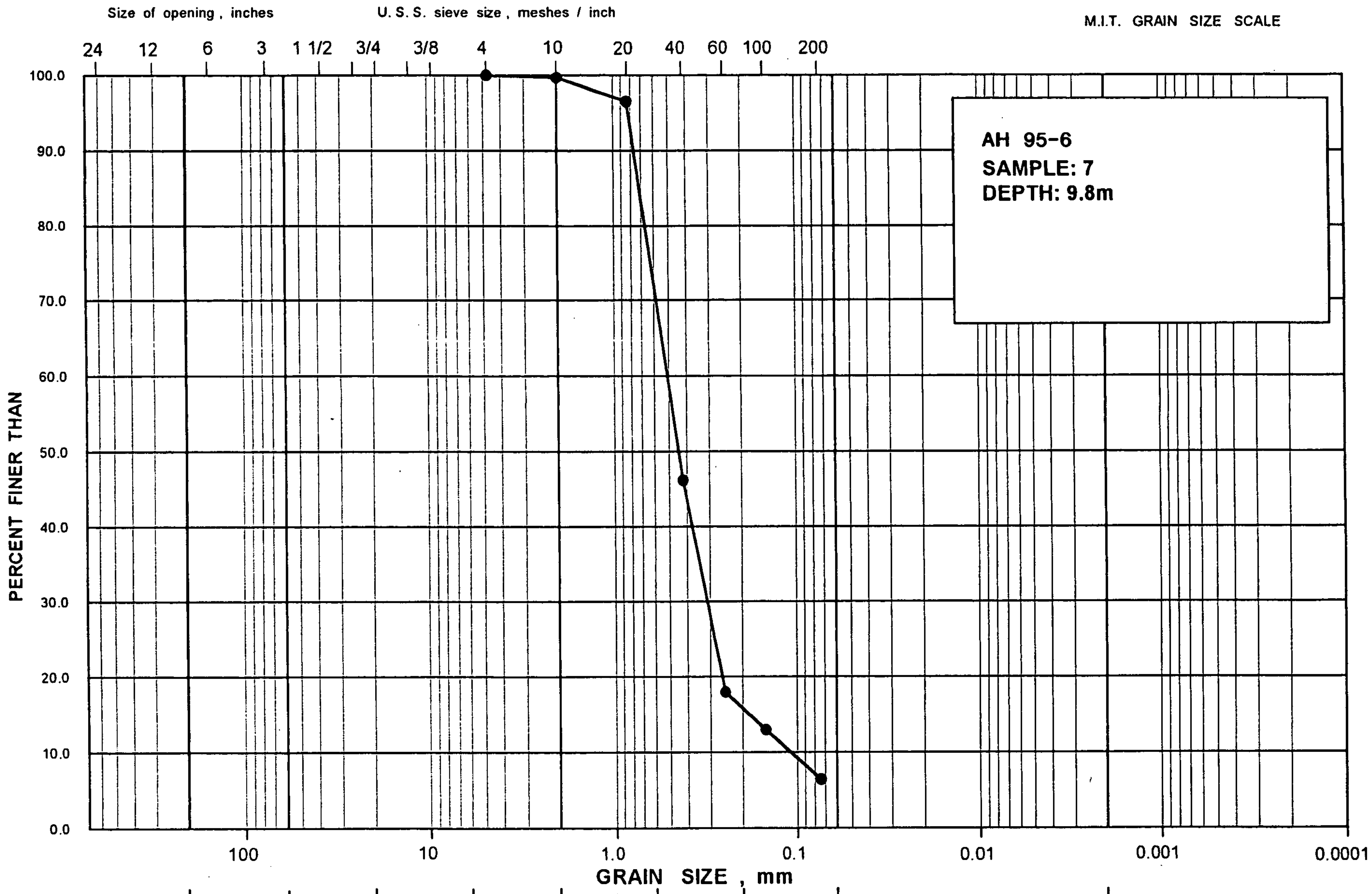
Project No. ..9421211A....
Drawn ....MP.............
Reviewed ....
Date ....03/13/95......

Golder Associates

GRAIN SIZE DISTRIBUTION

Figure

II–i



BOULDER SIZE	COBBLE SIZE	coarse	medium	fine	coarse	medium	fine	fine grained	
		GRAVEL SIZE			SAND SIZE			SILT SIZE	CLAY SIZE

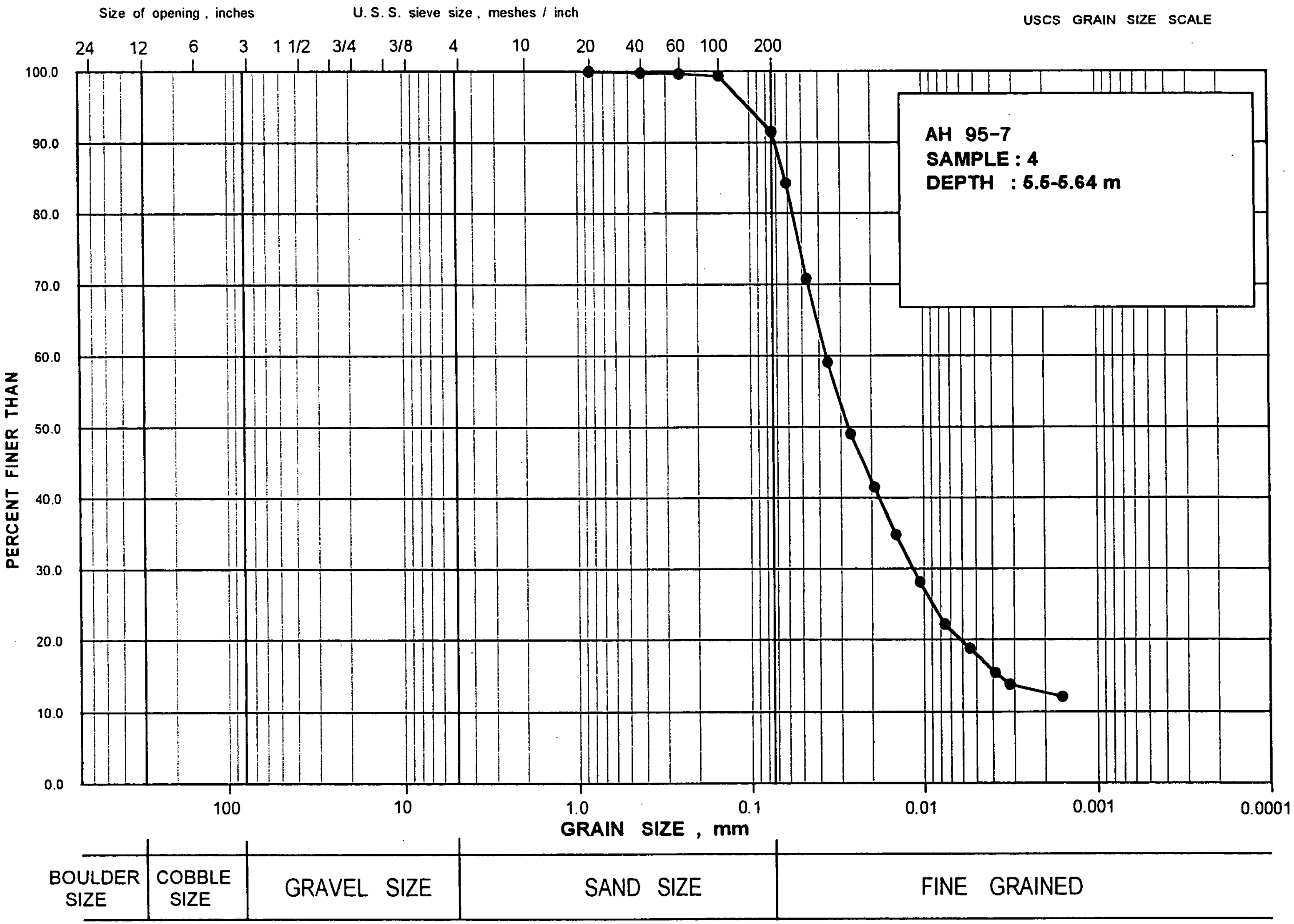
Project No. 9421211A
Drawn MP
Reviewed
Date 03/14/95

Golder Associates

GRAIN SIZE DISTRIBUTION

Figure

II–j



Project No. ..9421211A...
Drawn ...............M.O.......
Reviewed ....
Date ....03/30/95......

Golder Associates

GRAIN SIZE DISTRIBUTION

Figure

II–k