# Appendix WRM4-A

## Fracflow Technical Memo on Expansion of Town of Stephenville Potable Water Supply

world energy GH2

Fracflow Consultants Inc.
Environmental, Hydrogeological and Geotechnical Engineering Consultants

world energy GH2

# TECHNICAL MEMORANDUM

TO: David Pinsent, Kevin Boudreau — FFC-NL-3168-EIS-001
World Energy GH2 Limited Partnership

FROM: Fracflow Consultants Inc.

DATE: December 12, 2023

SUBJECT: Town of Stephenville, Potable Water Capacity Expansion

## 1.0 INTRODUCTION

The Town of Stephenville's water supply is provided by nine groundwater wells that are located in three distinct areas (**Figure 1**). Four wells (Well 7 to Well 10) are located on the east side and adjacent to Blanche Brook (just below where the Cold Brook tributary merges with Blanche Brook). Two wells (Well 2 and Well 4) are located south of the Hansen Highway. Three wells (Well 1, Well 5, and Well 6) are located north of the Hansen Highway near what is known locally as Beaver Pond. The water supply wells produce water from at least four aquifer systems, bedrock and overburden. All production wells are screened wells.

As with most groundwater wells, the well yields decrease over time due to plugging of well screens and/or dewatering of the aquifer. Fracflow completed redevelopment of five of the existing production wells (Well 2, Well 6, Well 7, Well 9, and Well 10) and replaced one well (Well 8) in 2023, which had a casing failure. The redevelopment work improved the well yields to the level needed to supply the Town of Stephenville. It is expected that several of the wells will have to be treated with acid to restore the full well yield.

The large bedrock well (Well 1) is scheduled to be reconstructed in 2024 due to a damaged casing but this well continues to produce approximately 325 USgpm. The Town also plans to add two new production wells in 2024.

As part of the aquifer protection, two monitoring wells (**Figure 1**) were installed in 2022 on the west side of Blanche Brook, between the Town`s water supply wells and the closed Stephenville landfill. These two monitoring wells are being monitored at regular intervals for water level, water temperatures and fluid conductivity with water samples being collected and analyzed annually. Starting in 2022, Leveloggers were installed in seven observation wells on the east side of Blanche Brook to monitor drawdowns in the four aquifer systems to determine if the reduction in well yields were due to aquifer dewatering or well screen issues.

## 1.1 Proposed Well Field Expansion

In 2021 Fracflow completed a 3D Hydrogeological Model of the Stephenville Well Field to outline capture areas, estimate aquifer capacity, calculate the impacts of the four wells that are located along Blanche Brook on the Blanche Brook baseflow and to define the well field protection areas and the Watershed Boundaries based on capture areas and travel times (Fracflow 2021). This report is attached as **Appendix 1**. The Fracflow 2021 report also includes the water budget or water balance calculation.

As noted, the 3D model was used to assess the current well field performance and the impact that withdrawals from the four wells that are located along the east bank of Blanche Brook are having on the baseflow in Blanche Brook. The 3D model was used to locate three new proposed production wells (FW1, FW2 and FW3) along the Cold Brook tributary of Blanche Brook. Fracflow completed several exploration wells along the Cold Brook tributary in the mid-1990s that indicated that moderate to high yielding production wells could be completed in this area. However, due to delays the Town of Stephenville experienced in obtaining access to the land for additional geotechnical drilling to finalize the locations and to design the production wells, two new production well locations (FW4 and FW5) were selected and evaluated using the 3D model.

One new production well (FW4) will be located south of Well 4, immediately north of Neds Pond and the second new production well (FW5) will be located north of Well 9, along the bank of Blanche Brook.  The drawdowns associated with these five new wells and all existing nine wells are provided in **Figure 2**. The particle tracks showing part of the pathways and the travel times for groundwater flow to these two new wells are shown in **Figures 3 and 4**.

These two new production wells are scheduled to be constructed and commissioned by the Town of Stephenville in the spring of 2024. These two new production wells, based on the well yield from nearby production wells, are expected to contribute an additional 400 USgpm to the Town’s water supply. The WEGH2 Limited Partnership construction camp, if connected to the Town’s potable water supply, is expected to require about 100 USgpd per person such that a 1,500 person camp would require approximately 110 USgpm over the project construction period.

# 2.0 REFERENCES

Fracflow Consultants Inc., 2021. Draft Report. 3D Hydrogeological Model of the Stephenville Well Field, Capture Areas, Aquifer Capacity, Impacts on Blanche Brook Baseflow and Watershed Boundaries, Stephenville, NL. Report FFC-NL-555-004. July 14, 2021. 130p.

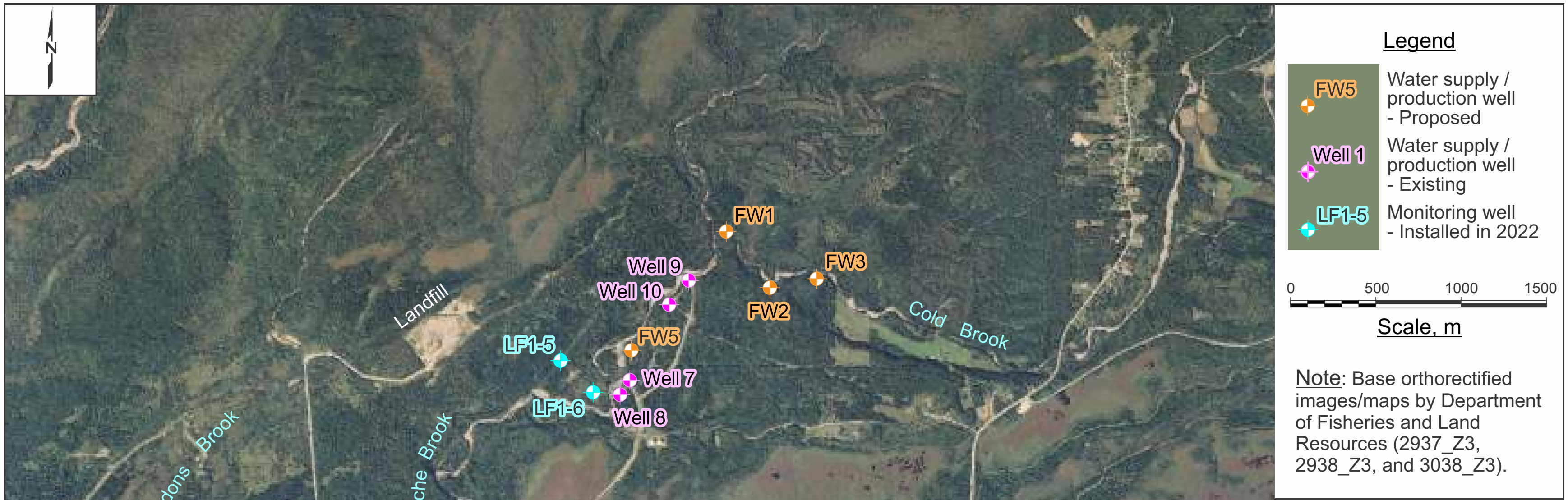


Figure 1

Location map of the nine water supply/ production wells and two monitoring wells (drilled in 2022) with five proposed production wells at the Town of Stephenville.

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| Document Reference | FFC-NL-3168-EIS |
| Location | Stephenville, NL |
| Project No. | 3168-1 |
| Date | December 2023 |

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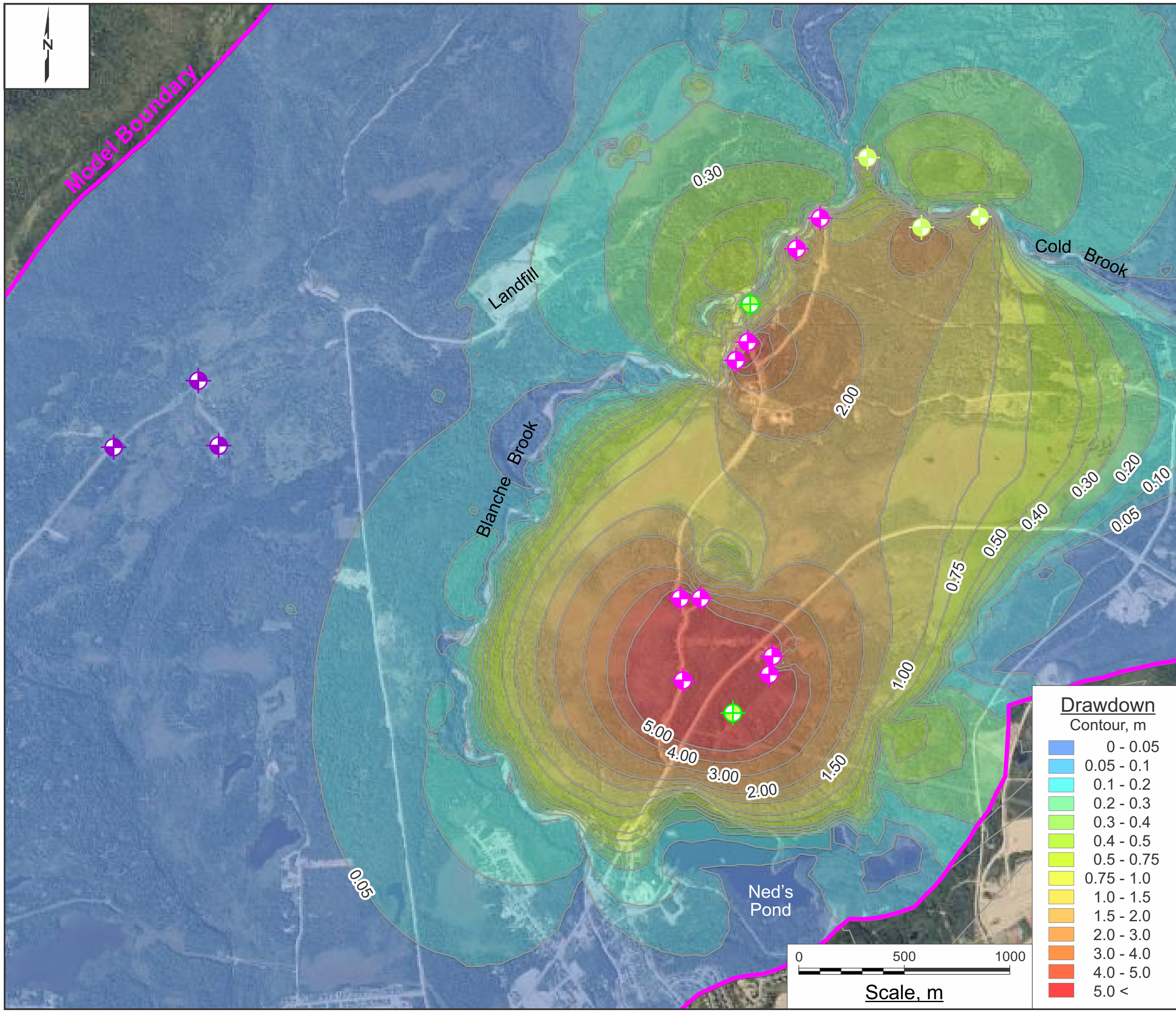


## Legend

- Production well under pumping condition - Stephenville
- Production well under non-pumping condition - Kippens
- Proposed production well for simulation of additional water supply
- Proposed production well - alternative location

Note: Base orthorectified images/maps by Department of Fisheries and Land Resources (2937_Z3, 2938_Z3, and 3038_Z3).

Figure 2

Capture area predicted by the model with steady-state drawdown contours in metres for a scenario of pumping five proposed wells as well as 12 existing wells from the Stephenville well field.

| | |
|---|---|
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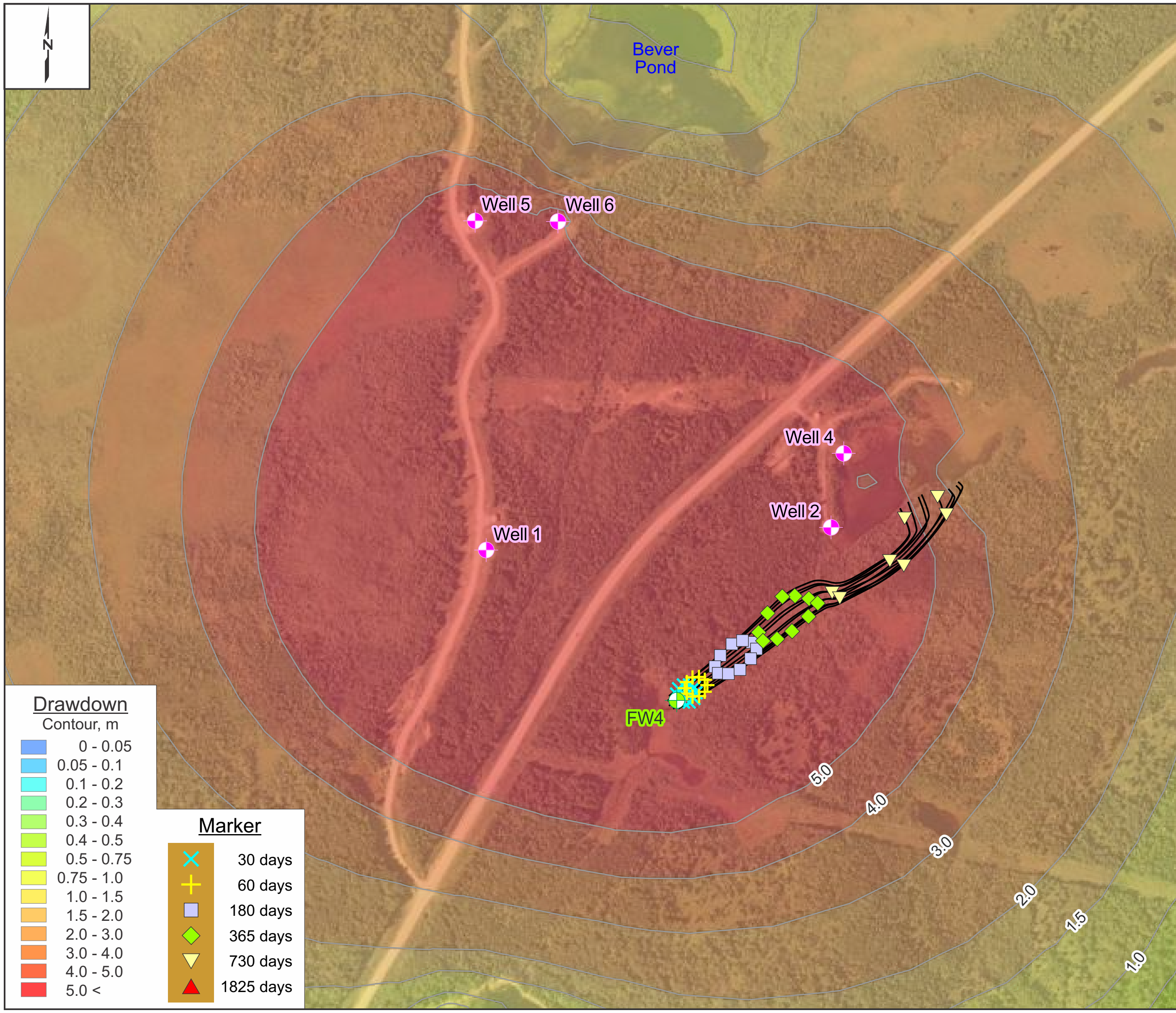


Legend

FW4 Production well under pumping condition - Proposed

Well 1 Production well under pumping condition - Existing

0 100 200

Scale, m

Note: Base orthorectified images/maps by Department of Fisheries and Land Resources (2938_Z3 and 3038_Z3).

Figure 3

Reverse particle tracks showing selected pathways for FW4 with markers for distance travelled between 30 days and 1825 days (5 years) with drawdown contours in metres for a scenario of pumping the five proposed wells as well as the nine existing wells.

Document Reference

FFC-NL-3168-EIS

Location

Stephenville, NL

Project No.

3168-1

Date

November 2023

FFC

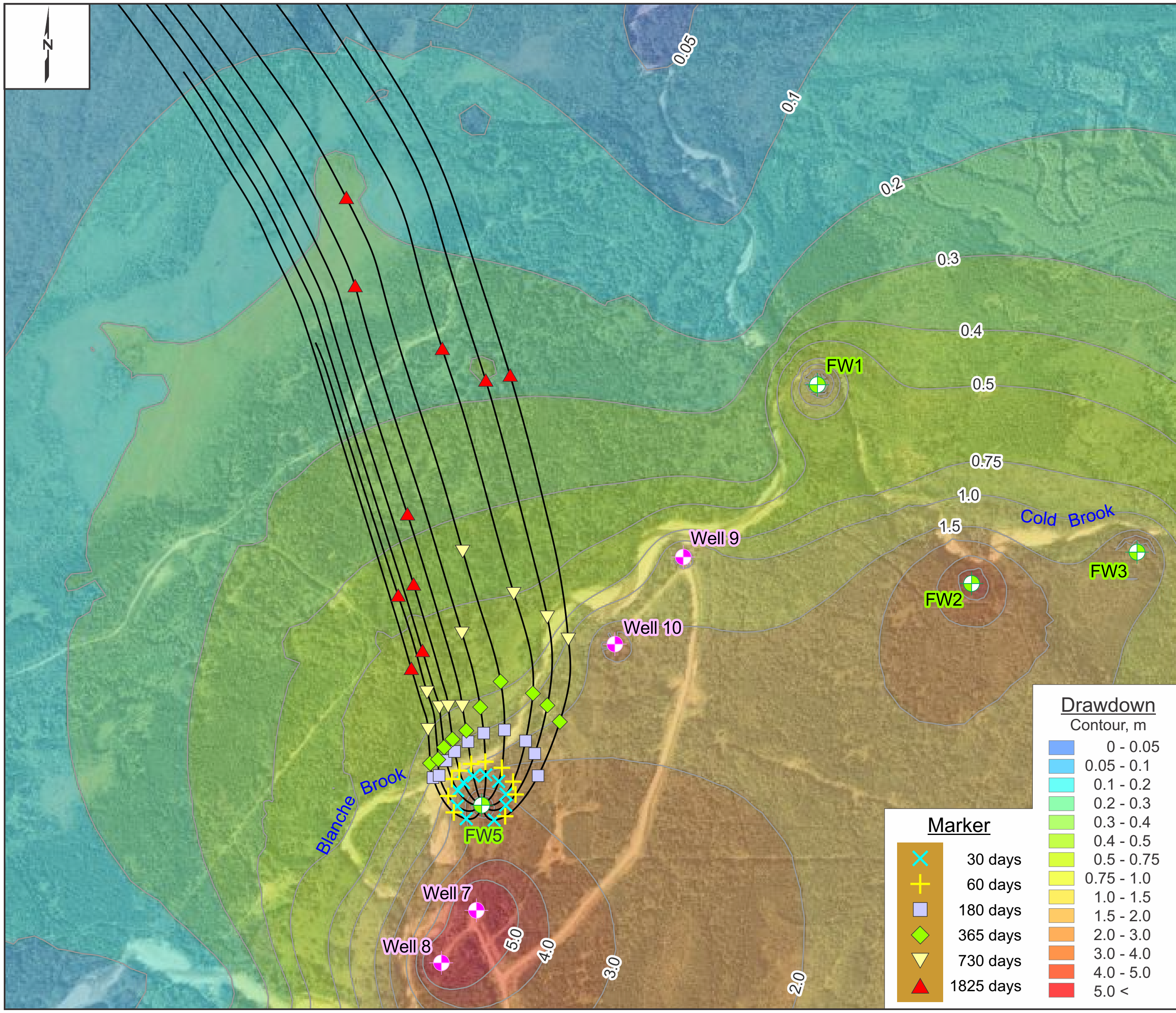


Figure 4

Reverse particle tracks showing selected pathways for FW5 with markers for distance travelled between 30 days and 1825 days (5 years) with drawdown contours in metres for a scenario of pumping the five proposed wells as well as the nine existing wells.

Document Reference
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Location
Stephenville, NL

Project No.
3168-1

Date
November 2023

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## *APPENDIX 1*

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***Draft Report FFC-NL-555-004***

***3D Hydrogeological Model of the Stephenville Well Field, Capture Areas, Aquifer Capacity, Impacts on Blanche Brook Baseflow and Watershed Boundaries***

Fracflow Consultants Inc.
Environmental, Hydrogeological and Geotechnical Engineering Consultants

Stephenville

Draft Report

# 3D Hydrogeological Model of the Stephenville Well Field, Capture Areas, Aquifer Capacity, Impacts on Blanche Brook Baseflow and Watershed Boundaries

*(FFC-NL-555-004)*

Prepared by:

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PROVINCE OF NEWFOUNDLAND
PERMIT HOLDER
CLASS "A"
This Permit Allows
FRACFLOW CONSULTANTS INC.
To practice Professional Engineering in Newfoundland and Labrador.
Permit No. as issued by APEGN D0169 which is valid for the year 2021.

July 14, 2021

REGISTERED PROFESSIONAL ENGINEER
JOHN E. GALE
SIGNATURE
14 July 2020
DATE
PROVINCE OF NEWFOUNDLAND

# Executive Summary

The Town of Stephenville's water supply is provided by nine groundwater wells that are located in three distinct areas. Four wells are located on the east side and adjacent to Blanche Brook (just below where the Cold Brook tributary merges with Blanche Brook), two wells are located south of the Hansen Highway and three wells are located north of the Hansen Highway near what is known locally as Beaver Pond. One of the water supply wells, a high capacity well, near Beaver Pond was completed in the bedrock aquifer. The other eight water supply wells were completed as screened wells in the overburden aquifer with those wells extending into the shallow bedrock with the well yield being derived primarily from the overburden. The Town of Kippens constructed four water supply wells in the bedrock aquifer to the west of the Stephenville well field and this is considered to be the same bedrock aquifer in which the high volume production well was constructed for the Town of Stephenville. The actual volume of groundwater being withdrawn from the bedrock aquifer has not been well documented and may exceed the bedrock aquifer capacity in the Kippens and Stephenville well field areas and may impact the available drawdown in the large capacity well in the Stephenville well field.

The four Stephenville wells that are located adjacent to Blanche Brook obtain part of their well yield through induced infiltration from Blanche Brook with potential impacts on baseflow in the brook during low flow periods. The initial groundwater flow system analysis, at the time the well field was being developed, was completed using a 3D finite difference model with a coarse mesh and without the benefit of the long term withdrawal of groundwater from all nine production wells. The original or initial watershed boundaries were extended to account for the lack of data on groundwater travel times and pathways. To refine the watershed boundaries, and to provide a basis for developing a revised well field protection plan that includes the simulation of different operational scenarios for the existing pumping wells, a new Stephenville groundwater model was constructed using the finite element model, FEFLOW. This model is more suited to simulate multiple production and observation wells because the area around each well can be sub-divided locally to evaluate the impacts of groundwater withdrawals on baseflow in Blanche Brook, to delineate watershed boundaries based on groundwater travel times, to assess other potential water supply areas, and to assess risks to the Town's water supply.

The 3D FEFLOW model was calibrated to the current conditions of the groundwater flow system in the well field, and used to evaluate the impact of potential contaminates such as the leachate from the abandoned and active landfills and proposed residential developments and other land use activities. The transport part of the FEFLOW model was used to calculate travel times from potential contaminate source areas to individual pumping wells. The revised wellhead protection plan can be based on the computed travel times and the capture areas for each water supply well. The existing monitoring well program for hydraulic head measurements and water quality has also been evaluated and the need for additional monitoring wells has been identified.

The 3D finite element FEFLOW was extended to capture the main components of the Blanche Brook and Gadon's Brook drainage basins, the areas in which the existing Stephenville and

Kippens production wells are located, and the areas in which additional production wells could be located. Calibration of the 3D well field model was completed using historical aquifer test data and current well field performance data as well as measurements of production well drawdown and water levels in available observation wells and the historical and current climatic and hydrology data to estimate groundwater recharge and baseflow conditions in Blanche Brook.

***Assessment of Impacts to Baseflow in Blanche Brook from Groundwater Withdrawals***

Runoff data for Blanche Brook are limited and do not extend over a significant time period. However, a much longer stream flow data set exists for Harry's River which, while having a much larger drainage basin, is characterized by similar terrain, overburden and bedrock. The Harry's River data were used to generate a baseflow recession curve for Blanche Brook to estimate the statistical, frequency and duration of low baseflow conditions.

The reliable yield for assessment of risk of water supply well induced infiltration withdrawals exceeding baseflow conditions may be taken as the 7-day, 10-year ($Q_{7,10}$) low flow. Blanche Brook, as calculated by comparison with Harry's River based on drainage basin areas and using Harry's River data, had a $Q_{7,10}$ low flow of 0.397 $m^3/s$. The flows observed in the flow distribution curve show that on any given day, the probability of the $Q_{7,10}$ flow of 0.397 $m^3/s$ being exceeded is 97.3%. The 3D model simulations show that the existing four water supply wells that are located adjacent to Blanche Brook produce a reduction in baseflow that is a small percentage of the computed low flow conditions. Additional water supply wells that are located along the Cold Brook tributary can be expected to withdraw a similar volume of water from the adjacent brook.

***Redefining Well Field Protection Plan and Watershed Boundaries Based on Five Year Travel Times.***

The 3D model simulations show that the Town should consider using the limits defined by the five-year travel time plots as the basis for revising the well field protection plan and adjusting and ranking the watershed boundaries. These revisions would protect the existing well field and future areas for new water supply well construction. The goal would be to ensure that the well field protection plan, and the revised watershed boundaries, reflect current and future water supply needs and land use preferences for the Town of Stephenville.

The 3D model flowpaths and travel time calculations also show that the Stephenville landfill poses a threat to at least Well 7 and Well 8 and at least three monitoring wells should be constructed between the Stephenville landfill and the west bank of Blanche Brook to monitor any potential leachate migration towards Well 7 and Well 8 and also Well 10.

***Assessment of Risks from Residential Systems in the Hillier Avenue Area***

3D model simulations were completed at three locations along Hillier Avenue to determine if residential land use activities that include septic tank systems and shallow overburden wells would impact the underlyng bedrock aquifer. Those simulations show that any wastewater that was released at those three locations would migrate down-gradient, partly to local ponds, and would not migrate more than 15 m vertically into the overburden over time. For most waste water releases, natural attenuation degrades and bioremediates the waste water as it migrates over distances that can be measured in 10’s of metres. Those releases do not appear to pose any risk to the underlying bedrock aquifer except in those cases where surface casing for wells that have been or will be completed into the bedrock aquifer were not fully sealed from the bottom to the ground surface.

***Well Field Management and Maintenance***

To minimize the impact of the existing and future water supply wells on baseflow conditions in Blanche Brook during sustained periods of low or no rainfall, consideration should be given to adopting a well field management plan that reduces withdrawals from wells adjacent to Blanche Brook and increases withdrawals from the remaining five water supply wells for those short periods.

The current well yields for the existing Stephenville water supply wells are generally lower than the original production rates that were estimated from the original aquifer tests. The production wells, except for the main bedrock well - Well 1, were constructed using a K-packer assembly. For wells with a K-packer construction, a gravel pack cannot be installed around the well screen and a natural gravel/filter pack is developed by both the normal well development procedures and by long-term pumping with repeated on and off cycles in each well. The object is to remove the fines from the aquifer that is in immediate contact with the well screen, to maintain permeability, and reduce well loss. However, over time with sustained pumping the well screen becomes partially clogged by fines and in some cases by bacterial growth on the well screen producing a reduction in specific capacity and loss of well yield.

Well field maintenance requires that screened wells be inspected on a regular basis by calculating the current specific capacity of each well to identify those wells where the specific capacity is lower than the well’s original specific capacity. In addition, the aquifer capacity has to be evaluated by measuring the static water level in each well to identify those wells in which the well withdrawals are exceeding the local aquifer capacity, taking into account any well interference impacts on water levels from any nearby active production wells. Note that during the original aquifer tests, the drawdowns or well interference in nearby wells were measured and recorded.

For wells that show a reduction in specific capacity, but no significant reduction in static water levels, a Biological Activation Reaction Tests (BARTs) test should be performed. If the BARTs test does not show any obvious bacterial growth in the well, it is reasonable to assume that the reduction in specific capacity is due to an accumulation of fines around the well screen. To remove the fines from around the well screen, an aggressive program of well re-development using simultaneous surging with a surge block and air-lifting to remove the accumulated fines needs to be undertaken on a regular basis followed by measurement of the specific capacity of each well. Restoring or improving the specific capacity of productions wells, in the absence of major reductions in static water levels, is the most cost-effective approach to increasing or restoring overall water supply.

To determine the current static water level for existing wells, each well has to be shut down for a 24-hour to 48-hour period with continuous monitoring of the water levels, with a five-minute measurement interval, in each shut-in well and in each nearby pumping well and observation well. The goal is to determine if the aquifer capacity is being exceeded by excessive long term aquifer withdrawals. Those data will inform the Town of the need for additional wells and provide guidance on where any new production wells are best located.

It is recommended that any new production wells should be constructed, once the aquifer geology has been established, by diamond drilling with packer testing at the proposed production well location, by first driving a 300 mm (12-inch) casing to bedrock or to the planned well depth, then assembling and placing a 200 mm (8-inch) well screen and casing assembly in the 300 mm (12-inch) casing, followed by placing a silica sand filter pack around the well screen as the 300 mm (12-inch) casing is withdrawn with placement of a standard bentonite well seal and concrete collar above the well screen and at the top of the well. Once the well has been constructed, the normal sequence of well development using surging and air-lifting needs to be completed to settle the silica sand filter pack. This design, using an artificial filter pack, while more expensive to construct, is expected to reduce the frequency of well re-development and overall well maintenance.

***Potential Locations for Additional Water Supply Wells***

The 3D model simulations show that the Town cannot develop additional bedrock wells in the area at the northeast end of Beaver Pond. Also, the model simulations and the current static water levels suggest that this bedrock aquifer is being over-exploited. The addition of a fourth well in the Kippens' well field without any obvious or known assessment of the long-term impact on the common bedrock aquifer may have contributed to the overall decrease in water levels in the bedrock aquifer.

The recommended location for constructing one or more production wells is at and east of the point where the Cold Brook tributary joins Blanche Brook. Three potential water supply well locations and the expected drawdowns for well withdrawals of 1,000 L/min (264 USgpm) were computed. Water supply wells that are developed in this area would be located approximately 250 to 300 m from the end of the pipeline for Well 9. Also, the impact on baseflow in Blanche Brook was computed and those impacts are consistent with the impacts from the existing production wells. Construction of one or more water supply wells will require road access and a geotechnical drilling site investigation at each location to determine how close the wells can be placed to the Cold Brook tributary or to Blanche Brook.

Additional production wells can be constructed east of this location along the edge of the farm property adjacent to the Cold Brook tributary but each production well will impact, to some extent, the baseflow conditions in the Cold Brook tributary and hence in Blanche Brook. In addition, developing water supply wells in that area will significantly increase costs for pipelines and power supply.

The other major aquifer in the Stephenville area is the high capacity granular aquifer which currently provides the water supply for the NHSL fish hatchery. This aquifer is located under Warm Creek and to the east of Warm Creek. Wells developed in this aquifer have the capacity to produce approximately 1,890 L/min (500 USgpm). A production well that is developed on the west side of Warm Creek will have limited impact on the overall yield from this granular aquifer.

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# 1.0 INTRODUCTION

## 1.1 Background

A hydrogeological investigation was carried out by Fracflow Consultants Inc. between 1996 and 1999 to determine the presence, extent and transmissive properties of aquifers in the immediate Blanche Brook area, both south and north of the Hansen Highway, and in the lower section of the stream branch that extends to Cold Brook. This field work was conducted to evaluate the quality and quantity of the groundwater, and evaluate the potential of the aquifer or aquifers to produce the groundwater supplies needed for the Town of Stephenville. Based on this field work, the locations of ten water supply wells were identified and the water supply wells were constructed. Nine of those production wells were commissioned (**Figure 1.1**).

Fracflow also completed a hydrogeological investigation for the Town of Kippens that resulted in the construction of three water supply wells in the early 1990s with a fourth production well being added by the Town of Kippens at a later date. The four production wells that supply the water for the Town of Kippens were constructed in the bedrock aquifer and this is considered to be the same bedrock aquifer in which a high volume production well had been constructed for the Town of Stephenville. Fracflow's assessment was that the bedrock aquifer in the area immediately north of the Town of Kippens could support the estimated groundwater production from three wells. However, with the addition of a fourth water supply, the actual volume of groundwater being withdrawn from the bedrock aquifer have not been well documented and may exceed the aquifer capacity in the Kippens area and may impact the available drawdown in the large capacity well in the Stephenville well field.

The balance of the water supply for the Town of Stephenville is supplied by screened water wells that are completed in the overburden and the shallow bedrock. Approximately four of the Stephenville wells obtain part of their well yield through induced infiltration from Blanche Brook with potential impacts on baseflow in the brook during low flow periods. A preliminary three-dimensional (3D) numerical flow and transport MODFLOW model was constructed for the entire Stephenville well field area during the 1990s initial site investigation and well field development to simulate the general groundwater flow system for the well field. The MODFLOW model was constructed to assess the general drawdown and flowpaths that would be generated by the Stephenville well field and to define the general watershed boundaries.

Since the MODFLOW model was a finite difference model that uses a regular grid mesh, for a large area with distributed wells such as exists for the Stephenville site it is difficult to develop a detailed mesh around each well in order to simulate travel times and pathways, exclusion zones, and the migration of local contaminants from the shallow surface overburden to the bedrock aquifer. A more detailed model mesh was required to propose adjusted watershed boundaries based on travel times and quantify impacts to the baseflow in Blanche Brook from induced infiltration to existing and future water supply wells.

To refine the watershed boundaries, and provide a basis for developing a revised well field protection plan that includes the simulation of different operational scenarios for the existing pumping wells, a new Stephenville groundwater model has been constructed using the finite element model, FEFLOW, which is a well-known commercial finite element flow and transport modeling software. Unlike the finite difference code, the finite element approach is more suited to simulate multiple production and observation wells because the area around each well can be sub-divided locally to capture near-wellbore aquifer response but expanded substantially away from areas of interest resulting in a significantly smaller number of nodal points which reduces simulation times. The FEFLOW model provides much more local detail than can be achieved by a finite difference model. In addition, the model area in the revised model has been extended to a broader area to capture the more extensive watershed boundaries and to enable the assessment of other potential water supply areas.

This new finite element model captures the details of the local and regional flow systems, such as recharge and discharge areas, to identify potential future water supply areas to support the Town's options for constructing additional water supply wells to supplement the existing production wells. The model, calibrated to the current conditions of the groundwater flow system in the well field, has been used to evaluate the impact of potential contaminates such as the leachate from the abandoned and active landfill and proposed residential developments and other land use activities. The transport part of the FEFLOW model was used to calculate travel times from potential contaminate source areas to individual pumping wells.

The revised wellhead protection plan should be based on the computed travel times and the capture areas for each water supply well. The existing monitoring well program for hydraulic head measurements and water quality has also been evaluated and updates have been recommended.

## 1.2 Objectives and Scope of Work

The objectives and scope of the project activities included:

1. Construction of a 3D finite element model using the FEFLOW software to capture the main components of the Blanche Brook and Gadon's Brook drainage basins, the areas in which the existing Stephenville production wells are located, and the areas in which additional production wells could be located. Calibration of the 3D wellfield model was completed using historical aquifer test data and current well field performance data as well as measurements of production well drawdown and water levels in available observation wells (**Figure 1.2**) and the historical and current climatic and hydrology data to estimate groundwater recharge and stream baseflow conditions.

2. Defining potential watershed(s) boundary(ies) and expanding the model domain to include new potential water supply areas based on travel times and groundwater flow pathways.

3. Estimating the impact of water well withdrawals on baseflow in Blanche Brook and Beaver Pond with reference to Blanche Brook low flow conditions and changes in water levels in or discharge from Beaver Pond.

4. Developing the data needed to produce a revised well field protection plan, based on computed travel times and capture areas and recommend adjusted effective watershed boundaries. Identify and evaluate areas of risk to the existing well field and to areas that can provide additional water supplies where baseflow impacts and land use options are acceptable. A revised protected watershed boundary based on five-year travel times can be designed to reflect current and future water supply needs and land use preferences for the Town of Stephenville.

5. Estimating risks to the bedrock aquifer from potential shallow overburden impacts.

## 1.3 Construction of Existing Production Wells

A brief discussion of the geological column intersected by each borehole and the well construction details, taken from the 1999 Fracflow Well Completion Report (Fracflow, 1999b), are presented below. The original production well numbering system is retained for this discussion with the revised well numbering provided in brackets in bold for each production well description sub-heading.

<u>Bedrock Well PTW-1</u> (Revised well number **Well 1**)

Well PTW-1 was drilled to a depth of 47.5 m in December 1997. During the 1998-99 well construction programme, a new 457 mm diameter surface casing was aligned over the existing casing and then driven to the bedrock. A total of 9.8 m of casing was driven to depth of 9.5 m. The smaller, 200 mm, casing was removed and the existing borehole was reamed to a new diameter of 438 mm to a depth of 45.1 m below ground surface. The bedrock at that site consisted of layered sequences of sandstone, mudstone and siltstone, as shown on the well log in **Appendix A**.

A length of 41.4 m of 300 mm diameter well screen assembly was lowered into the hole to a depth of 41 m below ground surface. The screen assembly consisted of a 1 m blank section of steel casing, with a bottom plate, followed by 15.4 m of stainless steel wire wrapped screen, slot 60, and 25 m of blank steel casing (riser pipe) on the top. Four centralizers were placed along the screen assembly at depths of 39.5 m, 30.8 m, 23.6 m and 13.9 m.

After the screen assembly was installed, the annular space between the well screen and casing and the borehole wall was filled with well graded, clean, washed gravel in the 3 to 8 mm size range. This gravel filter was installed from the top of the backfill, at 43 m depth, to a depth of 10.6 m below ground surface, which is 14 m above the top of the well screen.

A seal, consisting of a 2.5 m column of bentonite chips, was placed on top of the gravel pack, 1.2 m below the bottom of the surface casing and 1.3 m in the casing. A continuous grout seal, consisting of Portland cement with 4% bentonite, was installed from the top of the bentonite seal to the top of surface casing (0.3 m above ground surface). The total length of bentonite and cement/grout seal was 11 m.

Reaming and installation of the well PTW-1 was carried out from November 25 to December 11, 1998. Well installation and construction details and basic geological data are presented on the well log in **Appendix A**.

<u>Well PTW-2</u> (Revised well number **Well 5**)

Well PTW-2 was drilled on December 6 and 7, 1997, west of Beaver Pond, 15.75 m east of BB2. The well encountered bedrock at a depth of 34.1 m and was drilled to a total depth of 34.7 m. The bedrock encountered in the borehole consisted of red-brown mudstone and siltstone and grey sandstone. The 200 mm well casing was driven to a depth of 30.5 m.

The overburden material from surface to a depth of 6.1 m consisted mostly of coarse sand and gravel with some clay and cobbles, underlain from 6.1 to 9.4 m by brown, coarse, sand and angular gravel with clay. Below 9.4 m the overburden consisted of fine sand and clay with some gravel to 11.3 m and brown clay with some fine sand and gravel to 15.5 m, then grey, coarse sand, gravel and fine sand with traces of clay to 21.6 m. The lower overburden layers consisted of coarse sand and gravel with traces of clay to 23.8 m and fine, grey, sand with some gravel and clay to 24.7 m, underlain by grey clay with some gravel and fine sand to 26.2 m. The material above the bedrock from 26.2 m to 29.9 m consisted of angular gravel and coarse sand with some fine sand, clay and silt, then fine sand and clay, some coarse sand and gravel to 31.7 m, red-brown clay with fine sand to 33.2 m and coarse sand with gravel to 34.1 m.

A total of 9.4 m of 190 mm outside diameter, telescopic, well-screen assembly was installed in the well to a depth 33.9 m. The well-screen assembly consisted of a 3.2 m long blank section, black steel pipe with bottom plate followed by 5.1 m of 40 slot, stainless steel wire wrapped, well screen, 0.9 m stainless steel riser pipe and 0.2 m neoprene rubber K-packer at the top. The top of the screen assembly reached the depth of 24.5 m, or 24.9 m below the top of the casing (TOC). The casing was pulled back for 5.5 m, so the total length of casing left in the hole was 25.4 m with the bottom at 25.0 m depth and 0.4 m extending above the ground surface.

In 2020 the casing in this well failed and started allowing sand and gravel to enter the well just above the K-packer. The existing well was decommissioned by using alternating layers of granular bentonite and silica sand as per guidelines. A replacement Well 5 was drilled approximately 3 m away from the original well between November 14 and 15, 2021. A 200 mm diameter casing was advanced in 1.5 m increments to approximately 35 m below ground surface the overburden from approximately 1 m below ground surface (bgs) to approximately 15 m of depth consisted of fine to medium sand with some gravel. A 9.5 m layer of coarse sand and gravel was encountered at approximately 15 m of depth followed by a 3 m thick zone of fine sand with some silt/clay and then approximately 3 m of gray clay followed by a 4.5 m layer of brownish/red clay to 35 m bgs. The borehole was terminated when the design depth of 35 m bgs was reached.

Construction of the well assembly was preceded by pulling the 200 mm regular steel casing back to approximately 28.85 m bgs to avoid the thick clay layers at approximately 28.96 m bgs. Due to concerns that the stainless steel screen assembly would sink through the soft clay layer, clean crushed stone (fill) was added to the casing to fill the borehole while the casing was being pulled back to 28.85 m bgs.

The water well was constructed using a K-Packer assembly, consisting of a 0.93 m section of 152 mm stainless steel casing with a stainless steel plate welded to the bottom to act as a sand trap. A 6.25 m section of 40 slot screen was then attached to the sand trap with a 0.90 m section of 152 mm stainless steel casing between the well screen and the K-Packer. Once the well screen and K-Packer assembly had been lowered into place inside the 200 mm casing, the casing was pulled back approximately 7.62 m.

Construction and well installation details and basic geological data are presented on the well logs provided in **Appendix A**.

<u>Wells PTW-3 and PTW-4</u> (Non-production Wells)

Well PTW-3 was drilled on December 5 and 6, 1998 on the west side of Beaver Pond. Bedrock was encountered at a depth of 31 m below ground surface. The well was completed at a depth of 33 m. The bedrock encountered in the borehole consisted of red-brown and grey mudstone. The overburden material from surface to a depth of 4.3 m consisted of loose rounded gravel, coarse sand, clay and cobles, which was underlain by layers of silt, sand and clay with very little gravel. Some water (<50 L/min) was encountered on the top of bedrock in a 0.5 m thick layer of coarse sand and gravel with brown silt and clay. A total of 27.3 m of 200 mm casing was driven to depth of 27 m. No screen was installed in this well. The 200 mm casing was left in the hole to facilitate any future well development efforts.

Well PTW-4 was drilled between December 9 and 11, 1998 on the north side of Beaver Pond (**Figure 1.1**). The well encountered bedrock at a depth of 27.5 m and was drilled to a total depth of 33.0 m. The overburden material from surface to a depth of 5 m consisted of loose rounded gravel, sand, clay, and peat underlain by layers of silt, sand and clay with very little gravel. The bedrock consisted of poorly cemented conglomerate and sandstone between 27.5 and 30.5 m depth, underlain by grey and brown mudstone to the bottom of the well. Water was encountered as the borehole entered bedrock and the flow rate increased while drilling the upper 3 m of bedrock, reaching a maximum yield of 200 L/min. A total of 24 m of 200 mm diameter casing was driven to a depth of 23.8 m and was left in the borehole. No screen was installed in this well.

Construction and well installation details and basic geological data for both wells are presented on well logs provided in **Appendix A**.

Well PTW-5 (Revised well number **Well 2**)

Well PTW-5 was drilled on December 2 and 3, 1998 south of the Hansen Highway. The well encountered bedrock at a depth of 16.2 m and was drilled to a total depth of 20.7 m below ground surface. The bedrock consisted of red-brown mudstone and siltstone. A total of 15.2 m of 200 mm diameter casing was driven to a depth of 14.6 m.

The overburden material from surface to a depth of 3.0 m consisted mostly of black peat, followed by brown, fine sand with some silt and clay and traces of gravel to 6.0 m depth. This material was underlain by a layer of coarse sand, gravel and cobbles between 6.0 and 12.5 m depth, separated by a layer of fine sand and silt from 8.6 m to 10.0 m depth. The material directly overlying the bedrock, from 12.5 to 16.2 m depth, consisted mostly of grey and reddish silty clay.

A total of 6.6 m length of 190 mm OD, telescopic, well-screen assembly was installed to a depth 14.1 m. The well-screen assembly consisted of a 1 m blank section of black steel pipe with a bottom plate, followed by 4.7 m of 40 slot wire-wrapped well screen, 0.7 m of stainless steel riser pipe, and 0.2 m of neoprene rubber K-packer at the top. The top of the well-screen assembly reached a depth of 7.5 m below ground surface. The casing was pulled back for 6.5 m, so the total length of casing left was 8.7 m with the bottom at 8.3 m depth.

Well installation and construction details and basic geological data are presented on the well log provided in **Appendix A**.

Well PTW-6 (Revised well number **Well 4**)

Well PTW-6 was drilled between January 13 and 15, 1999, south of the Hansen Highway. The well encountered bedrock at a depth of 15.5 m and was drilled to a total depth of 18.8 m below ground surface.

The overburden material from surface to a depth of 4 m consisted of dark brown, coarse sand and clay with angular gravel and frequent cobbles and boulders, then brown-grey coarse sand with poorly rounded gravel and some fine sand, silt and clay to a depth of 7 m. This was underlain by rounded gravel and fine sand, silt and clay to 9 m, clayey silt with fine sand to 10 m, and fine to coarse sand with poorly rounded gravel, silt and clay to 13 m. The material above bedrock, from 13 to 15 m depth, consisted of brown-grey coarse sand and fine silty sand with some gravel, then compact, coarse sand and gravel to a depth of 15.5 m. The bedrock encountered in the wellbore consisted of coarse grained, grey sandstone and conglomerate.

A total of 15.2 m of 200 mm casing was installed to a depth of 14.7 m.

At a depth of 15 m to 15.5 m, water was encountered and a significant increase in flow rate with depth was noted while drilling through the bedrock. At a depth of 17 m the flow rate reached approximately 1,000 L/min. The well was drilled to a final depth of 18.8 m so the open screen interval could be placed over both the bedrock and overburden aquifers.

A total of 6.6 m length of 190 mm OD telescopic screen assembly was installed to a depth 18.8 m. The well-screen assembly consisted of a 1 m section of blank, black steel pipe with a bottom plate, followed by 4.7 m of 40 slot wire wrapped well screen, 0.7 m of stainless steel riser pipe and 0.2 m of neoprene rubber K-packer at the top. The top of the screen assembly reached a depth of 12.2 m below ground surface. The casing was pulled back 2.0 m. The total length of casing left in the hole was 12.8 m.

Well installation and construction details and basic geological data are presented on the well logs provided in **Appendix A**.

<u>Well PTW-7</u> (Revised well number **Well 3**)

Well PTW-7 was drilled on January 17, 1999, south of the Hansen Highway. The well encountered bedrock at a depth of 16.2 m and was drilled to a total depth of 17 m below ground surface. The bedrock encountered in the wellbore consisted of brown and red-brown mudstone.

The overburden material from surface to a depth of 1.3 m consisted of dark brown sand, fines and organics with rounded gravel and cobbles; very soft, dark-brown to black peat with traces of sand to a depth of 2.7 m; and rounded gravel with fines to a depth of 4.7 m. This material was underlain by angular gravel with coarse sand and some fine sand to 6.2 m; very soft clay, silt and fine sand to 9.5 m; and grey-brown rounded and angular gravel with coarse sand and some fine sand and fines to a depth of 12 m. The remaining overburden material overlying bedrock consisted of dark grey coarse sand with rounded and angular gravel and some fine sand and fines from 12 to 14 m; coarse and fine silty sand with poorly rounded gravel and some clay to a depth of 14.9 m; red-brown silt with traces of fine gravel to 15.9 m; and grey plastic clay from 15.9 m to a depth of 16.2 m that was in contact with the bedrock.

A total of 16 m of 200 mm diameter casing was installed to a depth of 15.7 m.

A total of 6.6 m length of 190 mm OD telescopic well-screen assembly was installed to a depth 16.3 m. The well-screen assembly consisted of a 1 m blank section of black steel pipe with a bottom plate, followed by 4.7 m of 40 slot wire-wrapped well screen, 0.7 m of stainless steel riser pipe, and a 0.2 m long neoprene rubber K-packer at the top. The top of the screen assembly reached a depth of 9.7 m below ground surface. The casing was pulled back 5.2 m, leaving 10.8 m of casing in the ground.

Well installation and construction details and basic geological data are presented on the well log provided in **Appendix A**.

Well PTW-8 (Revised well number **Well 6**)

Well PTW-8 was drilled on February 10 and 11, 1999, south of Beaver Pond. The well encountered bedrock at a depth of 22.8 m and was drilled to a total depth of 24.4 m below ground surface. The bedrock encountered in the wellbore consisted of brown, grey and grey-brown mudstone and siltstone.

The overburden material from surface to a depth of 3.0 m consisted of dark-brown gravel, coarse sand, some fine sand and clay, cobbles and boulders; followed by brown coarse and fine sand with some gravel to a depth of 5.0 m; and brown coarse sand with fine sand and some fines to a depth of 6.5 m. These materials were followed by brown gravel with coarse sand and some fine sand, fines and boulders to 8.5 m depth; brown coarse sand with some gravel and fine sand to a depth of 10.0 m; and brown clayey fine sand, some coarse sand and fine gravel between 10 and 12.5 m depth. The deeper overburden consisted of brown-grey coarse gravel, coarse sand and some fine sand to a depth of 16.0 m; dark grey silty sand and coarse sand with gravel and some clay from 16.0 to 21.3 m depth; and compacted, coarse sand with some fines to 22.4 m, and red-brown clayey silt to a depth of 22.8 m.

A total of 23.2 m of 200 mm diameter casing was installed to a depth of 22.9 m.

A total of 6.6 m length of 190 mm OD telescopic well-screen assembly was installed to a depth 22.5 m. The well-screen assembly consisted of a 1 m blank section of black steel pipe with a bottom plate, followed by 4.7 m of 40 slot wire-wrapped well screen, 0.7 m of stainless steel riser pipe, and a 0.2 m long neoprene rubber K-packer at the top. The top of the screen assembly reached the depth of 15.9 m below ground surface. The casing was pulled back for 6.4 m and a total of 17 m of casing was left in the ground.

Well installation and construction details and basic geological data are presented on the well log provided in **Appendix A**.

Well PTW-9 (Revised well number **Well 7**)

Well PTW-9 was drilled on February 16 and 17, 1999 in the Blanche Brook area, south of junction with Cold Brook. Bedrock was encountered at a depth of 30.4 m. The wellbore was completed at a final depth of 32.2 m. Bedrock consisted of red-brown mudstone and grey siltstone and sandstone.

The overburden material from surface to a depth of 9.7 m consisted of brown gravel, coarse sand, some fine sand and traces of clay, cobbles and boulders. A layer of brown silt with sand and gravel was intersected between 9.7 m and 11.1 m, followed by angular brown gravel with sand and silt to 12.5 m depth. This material was underlain by grey-brown, plastic silty clay with traces of gravel and sand to 19.4 m, and then brown-grey clay and silt with gravel and coarse and fine sand to a depth of 25.5 m. Between 25.5 and 28.0 m, a layer of brown-grey gravel and sand with fine sand, silt and some clay was encountered, followed by brown-grey clayey silt and silty clay with sand and gravel to bedrock.

A total of 30.8 m of 200 mm diameter casing was installed to a depth of 30.4 m.

A 6.6 m length of 190 mm OD, telescopic well-screen assembly was installed to a depth 31.1 m. The assembly consisted of a 1 m blank section of black steel pipe with a bottom plate, followed by 4.7 m of 40 slot wire-wrapped well screen, 0.7 m of stainless steel riser pipe, and a 0.2 m long neoprene rubber K-packer at the top. The upper part of the screen assembly was placed at a depth of 24.5 m below ground surface. The casing was pulled back 5.4 m, leaving 25.4 m of casing in the well.

After the casing was pulled back and cut, leaving a stickup of 0.4 m above the ground surface, water started to flow over the top of the casing. The flow rate was about 50 L/min and the water level rose to 0.42 m above the top of the casing, when the casing was capped. Before the pump testing was carried out in this well, a 0.6 m piece of 200 mm diameter casing was welded on the top of the well to prevent the uncontrolled discharge of water and to permit the monitoring of the static water level conditions.

Well installation and construction details and basic geological data are presented on the well log provided in **Appendix A**.

<u>Well PTW-10</u> (Revised well number **Well 8**)

Well PTW-10 was drilled on February 18 and 19, 1999 in the Blanche Brook area, approximately 150 m south of PTW-9. Bedrock was encountered at a depth of 32 m and the well bore was completed at a final depth of 33 m below ground surface. The bedrock consisted of red-brown mudstone.

The upper 4 m of overburden material consisted of brown, coarse gravel and sand, with some fine sand, clay, cobbles and boulders. A very soft clay occurred between 4 and 10.5 m depth, followed by a layer of brown sand and gravel with fines to a depth of 13 m. This material was underlain by a brown clay with some fine sand and traces of gravel to a depth of 14.7 m; a brown gravel with fine and coarse sand, clay and silt to a depth of 18 m; and a grey-brown clay and silt with some gravel and sand to a depth of 19 m. Between 19 and 26 m depth, the overburden consisted of grey clayey silt with traces of gravel and sand. The next 4 m of material was a grey-brown fine gravel and coarse sand with some fine sand and fines. A fine gravel and coarse sand with red-brown silty clay was present to a depth of 31.5 m, followed by dense red-brown clay with coarse sand and fine gravel overlying bedrock.

A total of 32.2 m of 200 mm diameter casing was installed to a depth of 31.6 m.

A 6.6 m length of 190 mm OD, telescopic well-screen assembly was installed to a depth 31.7 m. The assembly consisted of a 1 m blank section of black steel pipe with a bottom plate, followed by 4.7 m of 40 slot wire-wrapped well screen, 0.7 m stainless steel riser pipe, and a 0.2 m long neoprene rubber K-packer at the top. The top of the screen assembly was placed at a depth of 25.1 m below ground surface. As the casing was being pulled back, the screen assembly jammed and was pulled 1.2 m up the well bore. For this reason, the screen assembly had to be pulled out of the well.

The screen assembly was inspected for damage and a new K-packer was added on the top (total length of the screen assembly 6.8 m). A total of 31.8 m of casing was driven back to a depth of 31.4 m, the borehole cleaned out to 31.9 m and the screen assembly installed to this depth. The top of the screen assembly was placed at a depth of 25.1 m below ground surface. The casing was pulled back 5.5 m, leaving 26.4 m in the well bore.

After the casing was pulled back and cut, water flowed over the top of the casing, which was 0.5 m above ground level, at the rate of 65 L/min.

Well installation and construction details and basic geological data are presented on the well log provided in **Appendix A**.

<u>Well PTW-11</u> (Revised well number **Well 9**)

Well PTW-11 was drilled on March 17 and18, 1999 in the Blanche Brook area, approximately 500 m north of PTW-9. Bedrock was encountered at a depth of 22 m and the well bore was completed at a final depth of 24 m. The bedrock consisted of dark-brown mudstone.

The upper 4 m of overburden material consisted of 2.5 m of dark brown gravel, cobbles and boulders and some sand and clay; 0.3 m of brown sandy clay with some cobbles and gravel; and 1.2 m of fine angular and rounded gravel with some clay, sand and silt. This material was underlain by brown and grey silt and clay with traces of gravel to a depth of 16.7 m, and then angular gravel with coarse and fine sand and some clayey silt to a depth of 21.0 m. A 1 m thick layer of dark grey clayey silt with sand and some gravel was present immediately above the bedrock.

A total of 21.3 m of 200 mm diameter casing was installed to a depth of 21.2 m.

A 6.9 m length of 190 mm OD, telescopic well-screen assembly was installed to a depth 21.9 m below ground surface. The assembly consisted of a 1 m blank section of black steel pipe with a bottom plate, followed by 4.7 m of 40 slot wire-wrapped well screen, 1 m of stainless steel riser pipe, and a 0.2 m long neoprene rubber K-packer at the top. The top of the screen assembly was placed at a depth of 15 m below ground surface. The casing was pulled back 5.3 m, with the excess being cut off, leaving 16.3 m of casing in the hole.

After the casing was pulled and cut, water started to flow over the top of the casing which had a stickup of 0.4 m above ground. The flow rate was approximately 40 L/min. Before the aquifer testing was carried out on the well, a 0.9 m long piece of 200 mm diameter casing was welded on the top of the well to prevent uncontrolled flow of water and to establish static water level conditions. However, another 0.2 m long piece of casing had to be added to the top of the well before the artesian flow stopped.

Well installation and construction details and basic geological data are presented on the well log provided in **Appendix A**.

Well PTW-12 (Revised well number **Well 10**)

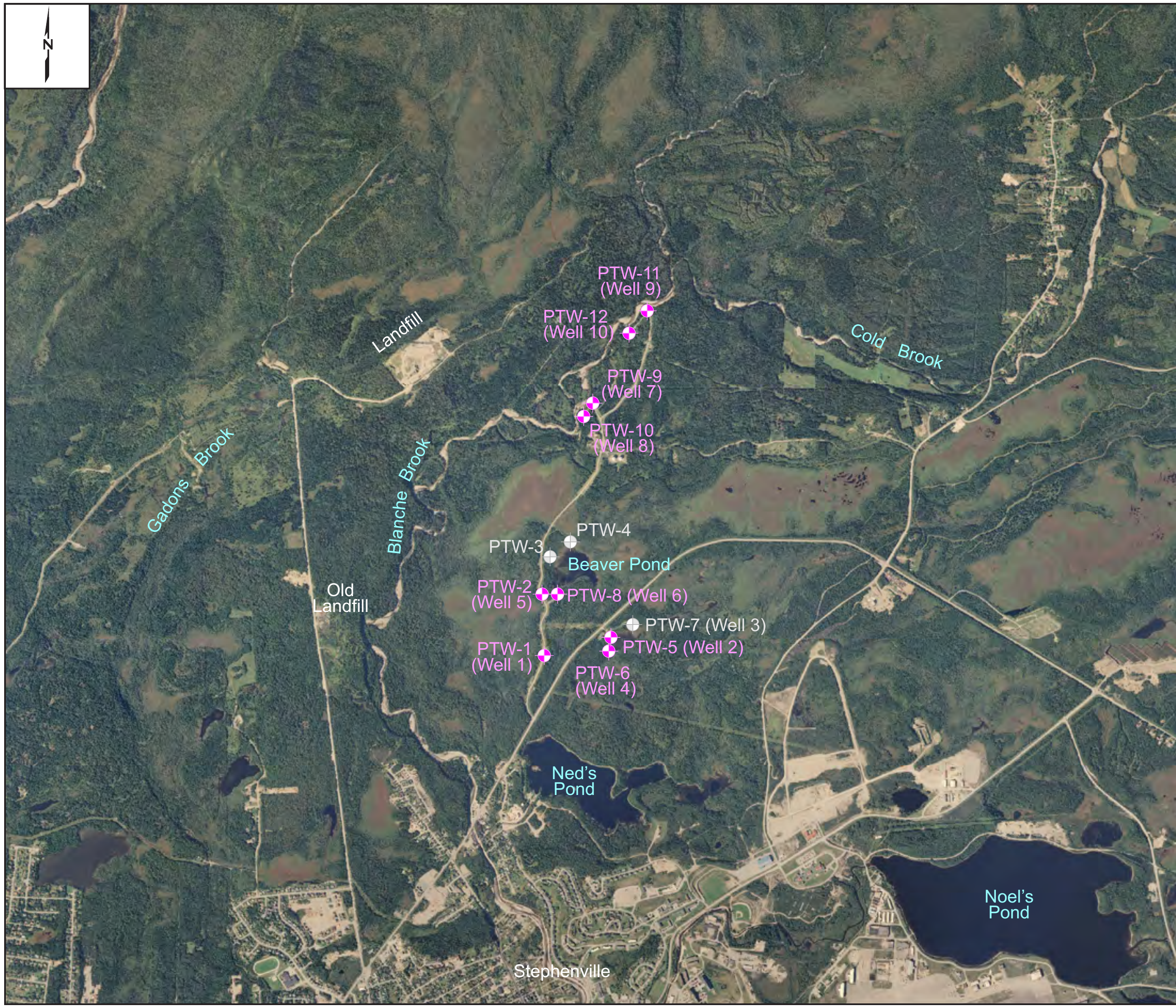
Well PTW-12 was drilled between March 24 and 27, 1999 in the Blanche Brook area, approximately 100 m south of PTW-11. Bedrock was encountered at a depth of 24.5 m and the well bore was completed at a final depth of 26 m below ground surface. The bedrock consisted of poorly cemented conglomerate and grey-brown mudstone.

The overburden material from surface to a depth of 11.5 m consisted of 4.2 m of brown, rounded gravel, sand, cobbles and boulders and traces of fines; 4.8 m of brown-grey clay with silt and traces of gravel and sand; and 2.5 m of grey-brown silt with clay and traces of gravel and sand. This material was underlain by brown and grey gravel and sand with silt and clay to 14.0 m; silty clay with sand and gravel to 15.0 m; and angular gravel with coarse and fine sand and some silt and clay to 20.0 m. The remaining 4.5 m of material overlying the bedrock consisted mostly of dense grey silt with clay and some gravel and sand.

A total of 21.2 m of 200 mm diameter casing was installed to a depth of 20.6 m.

A 6.8 m length of 190 mm OD, telescopic well-screen assembly was installed to a depth 20.9 m. The assembly consisted of a 1 m blank section black steel pipe with a bottom plate, followed by 4.7 m of 40 slot wire-wrapped well screen, 0.9 m of stainless steel riser pipe, and a 0.2 m long neoprene rubber K-packer at the top. The top of the screen assembly was placed at a depth of 14.1 m below ground surface. The casing was pulled back 5.9 m, with the excess being cut off, leaving 15.5 m of casing in the hole.

Well installation and construction details and basic geological data are presented on the well log provided in **Appendix A**.



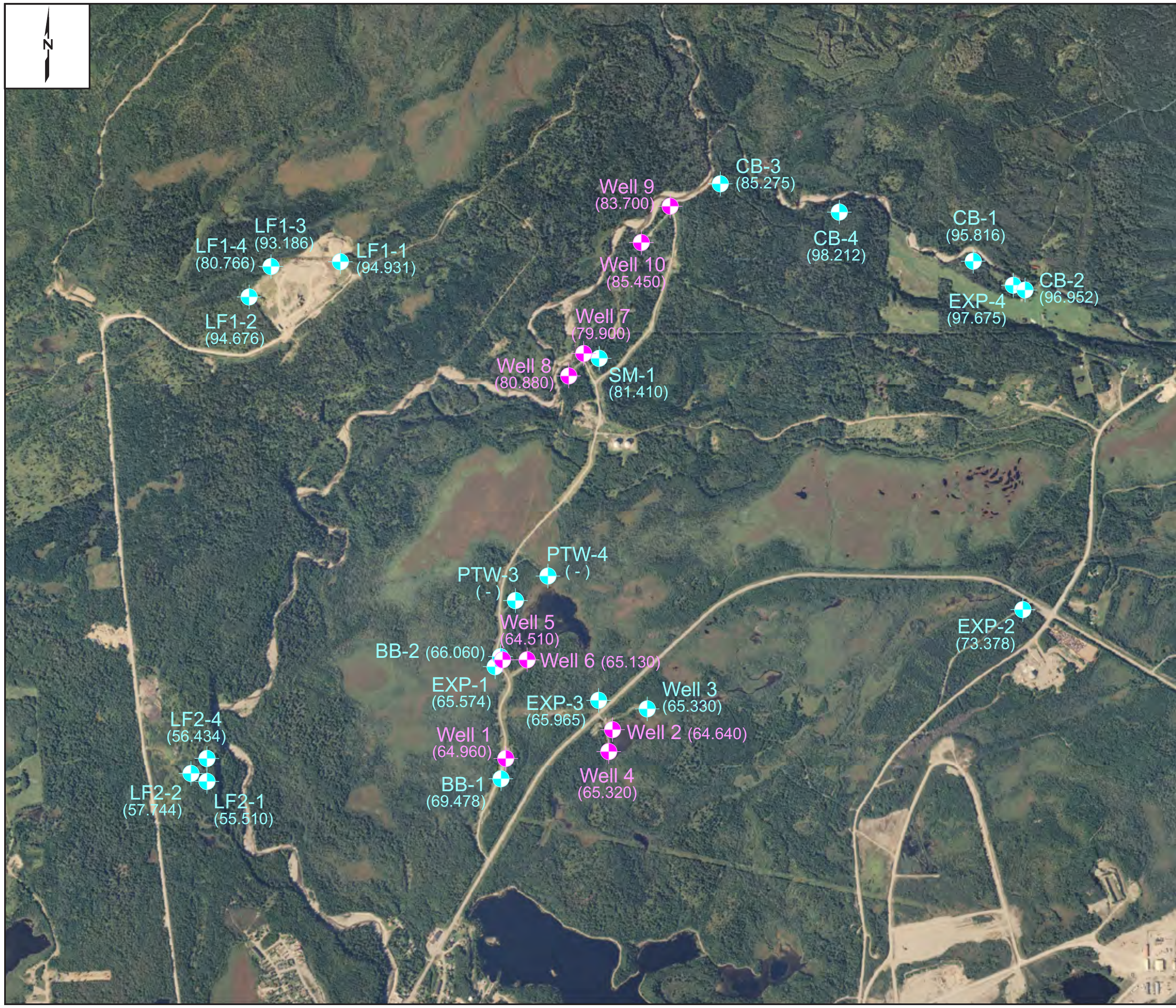

Note: Base orthorectified images/maps by Department of Fisheries and Land Resources (2937_Z3, 2938_Z3, and 3038_Z3).

Figure 1.1

Location map of the nine water supply/ production wells at the Town of Stephenville.

| | |
|---|---|
| Project No. | 555 |
| Location | Stephenville, NL |
| Document Reference | FFC-NL-555-004 |
| Date | June 2021 |

FFC




## Legend

Well 1 (64.960) — Production well with water elevation (m)

BB-1 (69.478) — Monitoring well with water elevation (m)

0 500 1000

Scale, m

Note: Base orthorectified images/maps by Department of Fisheries and Land Resources (2937_Z3, 2938_Z3, and 3038_Z3).

Figure 1.2

Production and monitoring wells with water elevation data compiled from the previous projects.

| | |
|---|---|
| Project No. | 555 |
| Location | Stephenville, NL |
| Document Reference | FFC-NL-555-004 |
| Date | June 2021 |

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# 2.0 GEOLOGY, CLIMATE AND HYDROLOGY

## 2.1 Drainage Basins and Topography

There are three main drainage basins within which the Stephenville well field is located, or adjacent to, with the Blanche Brook drainage basin being central to the Stephenville well field. Gadon's Brook drainage basin, located to the west of Blanche Brook, overlies the bedrock aquifer that supplies part of the water supply for the Town of Stephenville and all of the water supply for the Town of Kippens. Warm Creek drainage basin, which empties into Noel's Pond, is located to the east of Blanche Brook and has no apparent linkage or impact on flow in Blanche Brook. However, the large Warm Creek drainage basin does flow across the western boundary of the large overburden aquifer in which Northern Harvest Smolt Limited (NHSL) has developed their hatchery freshwater supply. The NHSL well fields are located to the south and east of Noel's Pond and the discharge from Warm Creek. The main recharge areas for the NHSL overburden aquifer are located east of Warm Creek and north to northeast of the existing NHSL well fields. Warm Creek is assumed to be a losing stream where it crosses over this overburden aquifer, based on the deep water table in this aquifer.

Blanche Brook in the area of the existing production wells forms a deeply incised valley and is considered to be the location of groundwater discharge which forms part or all of the Blanche Brook baseflow during low flow periods. Four of the existing Stephenville production wells intercept this groundwater discharge and reduce the baseflow during periods of low rainfall. The Blanche Brook drainage basin, at the point where it intersects the tributary from Cold Brook, extends north to areas with high elevation (**Figure 2.1**) of up to 320 m to 360 m that have exposed bedrock. This elevated area also provides some recharge to both the overburden and bedrock aquifer.

## 2.2 Geological Framework

Fracflow (1998) and Fracflow (1999b) summarized and reported on the surficial and bedrock geology from a series of field investigations and the sections of those reports are presented and summarized below.

### 2.2.1 Surficial Geology

Geological reports and maps (**Figure 2.2**) indicate that the Stephenville area is underlain by deposits of glaciofluvial sand and gravel ranging in thickness from 5 to 50 m. The overburden material consists mainly of poorly sorted, coarse sand and gravel with large cobbles. However, silty-clay layers have been identified deeper in the stratigraphic sequence at some locations and are present in most of the production well locations.

The depth to bedrock is variable in the Blanche Brook drainage basin. For example, two boreholes, BB1 and BB2, were drilled near Beaver Pond (east of Blanche Brook), 450 m apart. One borehole encountered bedrock at a depth of 4.6 m and the second borehole encountered 32 m of overburden above the bedrock.

In borehole BB1, the overburden material consisted of grey to grey brown sand with minor amounts of gravel and silt. Similar material was encountered in the upper section of borehole BB2. From about 9 to 23 m in BB2, the soil changed to denser multi-coloured sand with a large portion of gravel and more frequent cobbles and boulders. The material immediately above the bedrock, from 23 to 30.5 m, consisted of coarse sand and gravel with occasional cobbles and boulders. In this section, some piping sand was encountered in the drill rods and casing indicating that the sand should have high permeability. Just above bedrock, from 30.5 to 32 m, a layer of soft brown plastic clay was encountered. This clay showed increasing hardness with depth and appeared to grade into a brown consolidated mudstone which formed the top of the bedrock sequence in this area.

### 2.2.2 Bedrock Geology

The Blanche Brook drainage basin area is underlain by a diverse group of sedimentary bedrock types and structures (**Figure 2.3**). Bedrock immediately beneath the Stephenville area consists of a variety of carboniferous clastic sedimentary rocks of the Barachois and Codroy Groups, which consist predominantly of grey and red sandstone, with siltstone, mudstone, local conglomerate and minor coal beds. Black shale and carbonate (limestone and dolomite) exist to the west and north of the area of interest with contacts defined by major thrust faults. The original structure of the sedimentary rocks has been deformed, and extensive faulting and folding are evident within the various rock types (NFDOE, 1986).

The bedrock encountered in the general Kippens and Stephenville areas consists of layered sequences of mudstones, siltstones, sandstones and conglomerate. Many of the transmissive sections of the fractured porous sandstone which were cored were poorly consolidated and would easily break and crumble when handled (Fracflow, 1998 and 1999b).

## 2.3 Climate, Precipitation and Water Budget

As noted, the area from which the Town of Stephenville draws its groundwater supply is located within the Blanche Brook and Cold Brook drainage basin and the bedrock aquifer also extends west under the Gadon's Brook drainage basin. The overall Blanche Brook watershed is estimated to be approximately 126.43 km$^2$ (Acres, 1994).

**Figure 2.4** shows the monthly variations in total precipitation at the Stephenville Airport for the period of 1942 to 2020. The mean monthly precipitation varies from 67.60 mm (April) to 120.71 mm (December). Stephenville had a mean yearly precipitation of 1,226.76 mm between

1942 and 2020. The snowfall component (**Figure 2.5**) of the mean annual precipitation is 302.24 mm (equivalent rainfall) typically occurring between November and April with the highest monthly snowfall occurring in January (102 cm). **Figure 2.6** displays the historical annual precipitation values at the Stephenville Airport from 1942 to 2021. There are periods of low and high precipitation that tend to oscillate every five to ten years with a 30 to 40 year period of low precipitation (1942 to about 1970), increasing average precipitation between 1970 and 1985, followed by a period of higher but declining precipitation between 1985 and 2010. Overall, the recent trend appears to be one of decreasing annual precipitation.

The mean annual potential evapotranspiration for the area has been calculated to be approximately 500 mm per year (DOE, 1992). Calculations, by Fracflow, using the Stephenville International Airport weather records for the period of 1942 to 2007 and the Thornthwaite Equation, yield approximately 518 mm per year. The Thornthwaite equation tends to overestimate potential evapotranspiration which will lead to calculation of lower runoff estimates (Shaw, 1994).

As the Thornthwaite equation is dependent on average temperatures above freezing, it does not account for snow sublimation in winter months. Sublimation of snow can vary significantly from 5% to 50% of the snow pack. Sublimation is dependent on the groundcover, the latitude, the elevation, and climatic conditions. We have assumed sublimation accounts for precipitation loss in the study area by 10% during the months with average daily temperatures below freezing. On average, this is a loss of 41 mm of rainfall equivalent to sublimation per year.

**Figure 2.7** shows the temperature statistics at the Stephenville Airport for each month for the period of 1942 to 2021. The annual mean temperature for the area was about 4.93°C. The mean monthly temperatures were highest during July (16.21°C and August (16.42°C) and decreased to the lowest values during February (-6.42°C). The temperature statistics indicate that the mean monthly temperature between December and March is below 0°C. All of the climatic data have been obtained from Environment Canada websites.

Determining the sustainable long-term supply of groundwater for an area requires that the annual production rate (output) not exceed the rate of recharge (input) from precipitation within the catchment area of interest. Therefore, an assessment of the water balance within the Blanche Brook drainage basin was carried out with adjustments made to the normal assessment procedures to reflect that conditions that exist in different areas of the basin.

A water balance can be defined simply as,

$$P = R + E$$

where,

= Mean Annual Precipitation,
R = Mean Annual Runoff, and
E = Mean Annual Evapotranspiration.

Each of these components are defined and discussed separately below.

Determination of the surface runoff at the site is difficult to accurately assess without data collected in or near the drainage basin being studied. Typically, one would analyze hydrographs from gauged streams in the vicinity of the study area. Ideally these streams would have similar catchment areas and surficial geology to the study area. In this area, some but incomplete data are available for the Blanche Brook drainage basin. The other gauged streams in the area are Harry's River and Little Barachois Brook. The surface runoff data for Harry's River has been used for this analysis and by comparison to the ration of the drainage basin areas to develop the runoff response the Blanche Brook drainage basin to estimate baseflow conditions.

Preliminary studies conducted by the Newfoundland and Labrador Department of Environment on Little Barachois Brook indicate that groundwater recharge for that drainage basin was approximately 24% of the total precipitation (DOE, 1986). For the purposes of this analysis it is assumed that 24% of the precipitation input is contributing to deep and shallow groundwater recharge for areas that are not covered by marsh. The recharge calculated for the area is 294 mm.

<u>Average Generalized Water Budget (Expressed as Depths)</u>

| | | | |
|---|---|---|---|
| Input | = | 1,226 mm | |
| Output | = | 1,226 mm | |
| Precipitation | = | 1,226 mm | |
| Evapotranspiration | = | 518 mm | (42%) |
| Sublimation | = | 41 mm | (3%) |
| Recharge* | = | 294 mm | (24%) |
| Runoff | = | 373 mm | (30%) |

* Recharge to deep and shallow groundwater systems. Local topography and local hydraulic gradients in the overall area will result in some direct contributions to surface water.

The average generalized water budget presented above is useful in determining approximate volumes of water that will travel through a specific region during any given year. However, when assessing the risk of potential contaminant migrations and assessing concentrations / dilution factors, it is important to have an understanding of the seasonal fluctuations in the water budget. As such, Fracflow split the average generalized water budget into 12 months using average monthly data climate data for the Stephenville Airport from Environment Canada.

To complete this analysis, it was necessary to make some assumptions about frozen conditions in the winter months and the spring melt characteristics. To assess this, Fracflow examined the historical flow records that were reported for Harry's River and historical snowfall and snowpack data for the Stephenville Airport.

Using the flow records of the Harry's River, it was determined that the river system was typically in a base flow recession from December until the end of March. The size of the Harry's River drainage basin is 830 km$^2$ and encompasses portions of the Long Range Mountains; as such, portions of this basin will freeze up before, and melt after Stephenville has had its freeze and thaw periods. When analyzing the snow fall and snow pack data from Environment Canada for the Stephenville Airport, one can see a similar trend. These data show that typically there is snow pack recorded at the airport, starting at the end of December through to February or March where it will typically be gone by the end of April. Based on this data it was assumed that other drainage basins and sub-basins in the area would normally have no or little groundwater recharge or surface water runoff during January and February. Stream flow would be contributed primarily by groundwater discharge during those periods. Recharge would begin to occur again in March and April. The combined precipitation occurring in January and February is assumed to runoff or recharge in March and April with 50% occurring in each of those melt months.

However, bog covered areas within the Blanche Brook drainage basin do not freeze to any great depth and the marsh area over part of the basin area is assumed to be saturated with significant volumes of free water. This free water is available to recharge the underlying aquifer on a daily basis and is not affected by the surface temperatures, sublimation or evapotranspiration during the cold weather months.

The temperature and climatic records have been used to calculate the variation of recharge over the year for numerical model simulations in the following chapters of this report. The runoff data from Harry's River has been used to compute and estimate of baseflow conditions in Blanche Brook for comparison with the volume of water that is removed by induced infiltration to the production wells that are located adjacent to Blanche Brook.

## 2.4 Blanche Brook Discharge

Comparing the yearly precipitation data extracted for Harry's River and Blanche Brook (Acres, 1994), it is demonstrated that the ratios of observed yearly precipitation to average precipitation with respect to drainage area for the available data set are within reasonable agreement. Based on those comparisons, the data available for the Harry's River drainage basin may be used to compute discharge data for Blanche Brook by applying the ratio of the drainage basin areas to the recorded precipitation data (see Figure 2.7 to Figure 2.9 in the Acres report (Acres, 1994)). In the Acres report, the reported average drainage basin areas for Blanche Brook varied from 124.06 km$^2$ to 128.8 km$^2$ and the range of average values for the Harry's River drainage basin were 815.85 km$^2$ and 825.9 km$^2$ (based on Environment Canada, 1974 and Acres, 1994 data). The average of those values, 820.88 and 126.43 km$^2$, was used to convert flow data from Harry's River to flow data for Blanche Brook.

Comparing Blanche Brook to Harry's River, we can see a mean annual rainfall of 1,200 mm compared to 1,300 mm. If the various factors are weighted, a ratio of **7.0338** is determined for comparing Blanche Brook drainage basin annual rainfall to Harry's River drainage basin annual rainfall.

$$Q_{Harrys} * \left(\frac{Area_{Blanche}}{Area_{Harrys}}\right) * \left(\frac{MAR_{Blanche}}{MAR_{Harrys}}\right) \cong Q_{Blanche}$$

The observed flow values for Harry's River between August 1978 and March 1996 (excluding outliers) results in an observed ratio of **5.9280** for comparing Blanche Brook flows to Harry's River.

The weighted average of the two observed ratios based on physical comparison and observed flows (7.0338, and 5.9280, respectively) yields a ratio of **6.481**. This ratio was used to convert observed monthly average flow rates from Harry's River to monthly average flow rates for Blanche Brooke to obtain a flow distribution curve for Blanche Brook (**Figure 2.8**).

The ratio of monthly flows for each station was compared to the yearly mean to determine the months of low and high flows, as well as general trends and delays in local minima and maxima (if any). The ratio of monthly mean to yearly mean can be found in **Figure 2.9**. A number of outliers were observed in the data, where flow in Blanche Brook was significantly higher than Harry's River. Outliers were not included in the ratio calculation because they occurred when Blanche Brook flow is significantly higher than Harry's River (**Figure 2.9)** and would have exaggerated the flows calculated for Blanche Brook. Typically, Blanche Brook has a lower winter flow and higher summer flow. From **Figure 2.9** the observed winter flow has a ratio of 0.717 for the month of February, the lowest ratio observed (**Table 2.1**).

From the measured data (Environment Canada, 2007) the lowest recorded flow was 0.067 $m^3/s$ (17 March 1987) which was encountered during a 7-day average of 0.075 $m^3/s$ (11-18 March 1987), or a 30-day average of 0.133 $m^3/s$ from 19 February to 20 March 1987. The average monthly flow in 1987 for those months was 0.542 $m^3/s$ and 1.04 $m^3/s$ for Blanche Brook, compared to the average station monthly flow for February and March of 1.984 $m^3/s$ and 2.92 $m^3/s$, respectively.

The frequency analysis for Blanche Brook was completed by comparing the drainage area to the regression line on Figure 2.7 in the Acres report to obtain an 1-Day, 2-year ($Q_{1,2}$) low flow of 0.567 $m^3/s$ (**Table 2.2**). The $Q_{1,2}$ value was used as an index to relate Harry's River to Blanche Brook. The methods of probability analysis were not discussed in depth in the Acres 1995, report, so the values are assumed to be the product of the $Q_{1,2}$ low flow from drainage area, and some unit-less relation that incorporated return period, and low-flow duration.

The reliable yield, as discussed by Acres in the report (Acres, 1994, p2-28) may be taken as the 7-day, 10-year ($Q_{7,10}$) low flow. Blanche Brook, as calculated from Harry's River, had a $Q_{7,10}$ low flow of 0.397 $m^3/s$.

The flows observed in the flow distribution curve (**Table 2.3**) show that on any given day, the probability of the $Q_{7,10}$ flow of 0.397 $m^3/s$ being exceeded is 97.3%.

Table 2.1 Ratios of Mean Monthly Flow to Mean Yearly Flow for Blanche Brook and Harrys River.

| **Month** | **Monthly Mean / Yearly Mean** | | **Ratio** |
|---|---|---|---|
| | **Harrys** | **Blanche** | **Blanche/Harry** |
| January | 0.70 | 0.52 | 0.744 |
| February | 0.59 | 0.43 | 0.717 |
| March | 0.63 | 0.63 | 0.998 |
| April | 1.49 | 1.95 | 1.305 |
| May | 2.51 | 2.09 | 0.833 |
| June | 0.97 | 0.94 | 0.975 |
| July | 0.53 | 0.65 | 1.234 |
| August | 0.57 | 0.68 | 1.204 |
| September | 0.72 | 0.84 | 1.166 |
| October | 0.98 | 1.06 | 1.077 |
| November | 1.25 | 1.24 | 0.994 |
| December | 0.99 | 0.90 | 0.911 |

Table 2.2 Frequency Analysis Tables of n-Day, m-Year flows for Blanche Brook and Harrys River.

1-Day Low Flow

| Return Period | Harrys River 02YJ001 | Blanche Brook 02YJ002 |
|---|---|---|
| 2 Year | 4.368 | 0.567 |
| 5 Year | 3.052 | 0.396 |
| 10 Year | 2.489 | 0.323 |
| 20 Year | 2.104 | 0.273 |
| 50 Year | 1.761 | 0.229 |

7-Day Low Flow

| Return Period | Harrys River 02YJ001 | Blanche Brook 02YJ002 |
|---|---|---|
| 2 Year | 4.787 | 0.622 |
| 5 Year | 3.516 | 0.457 |
| 10 Year | 3.059 | 0.397 |
| 20 Year | 2.786 | 0.362 |
| 50 Year | 2.575 | 0.334 |

15-Day Low Flow

| Return Period | Harrys River 02YJ001 | Blanche Brook 02YJ002 |
|---|---|---|
| 2 Year | 5.465 | 0.710 |
| 5 Year | 3.961 | 0.514 |
| 10 Year | 3.408 | 0.443 |
| 20 Year | 3.070 | 0.399 |
| 50 Year | 2.805 | 0.364 |

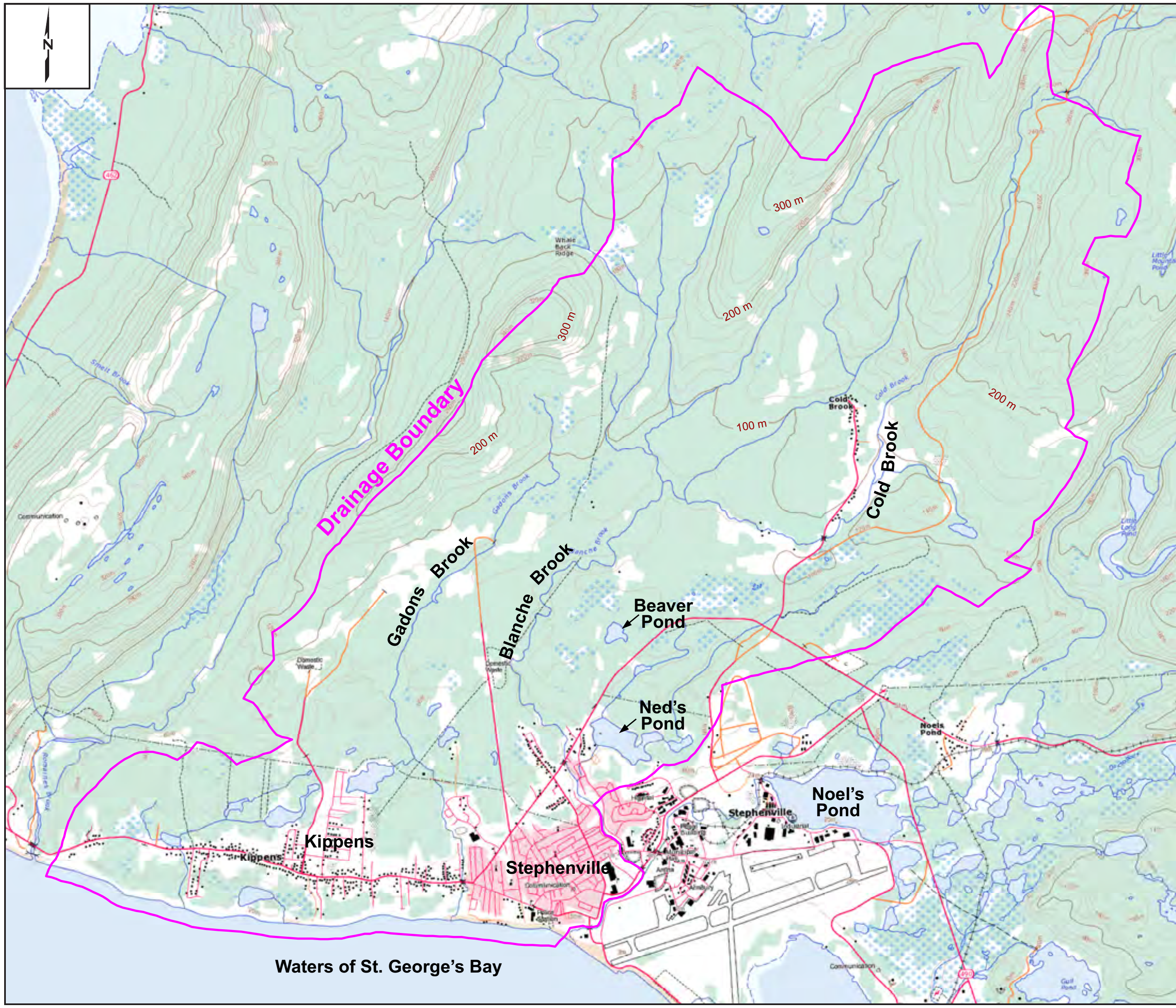
30-Day Low Flow

| Return Period | Harrys River 02YJ001 | Blanche Brook 02YJ002 |
|---|---|---|
| 2 Year | 6.532 | 0.848 |
| 5 Year | 4.707 | 0.611 |
| 10 Year | 4.071 | 0.529 |
| 20 Year | 3.698 | 0.480 |
| 50 Year | 3.418 | 0.444 |

Table 2.3 Daily Flow Duration Curve Observed Exceedances for Flows in Blanche Brook (1978-1996) and calculated flows from ratio of Harry's River (1968-2019).

| **Flow $m^3/s$** | **Exceed %** |
|---|---|
| 0.670 | 100 Min |
| 1.166 | 95 |
| 1.431 | 90 |
| 1.789 | 80 |
| 2.006 | 75 |
| 2.238 | 70 |
| 2.732 | 60 |
| 3.164 | 50 |
| 3.873 | 40 |
| 4.660 | 30 |
| 5.154 | 25 |
| 5.684 | 20 |
| 7.792 | 10 |
| 10.786 | 5 |
| 17.590 | 0 Max |

| **Flow $m^3/s$** | **Exceed %** |
|---|---|
| 0.067 | 100 Min |
| 0.499 | 95 |
| 0.650 | 90 |
| 0.989 | 80 |
| 1.169 | 75 |
| 1.379 | 70 |
| 1.839 | 60 |
| 2.399 | 50 |
| 3.189 | 40 |
| 4.460 | 30 |
| 5.290 | 25 |
| 6.509 | 20 |
| 10.399 | 10 |
| 15.799 | 5 |
| 62.900 | 0 Max |




0 1 2 3

Scale, km

Note: Base topographic map by Natural Resources Canada, 1/50,000: Stephenville, NTS 12 B/10, 2000 and Harris River, NTS 12 B/9, 1986.

Figure 2.1

Topographic map of the project site with drainage boundaries, Stephenville, NL.

| | |
|---|---|
| Project No. | 555 |
| Location | Stephenville, NL |
| Document Reference | FFC-NL-555-004 |
| Date | June 2021 |

FFC

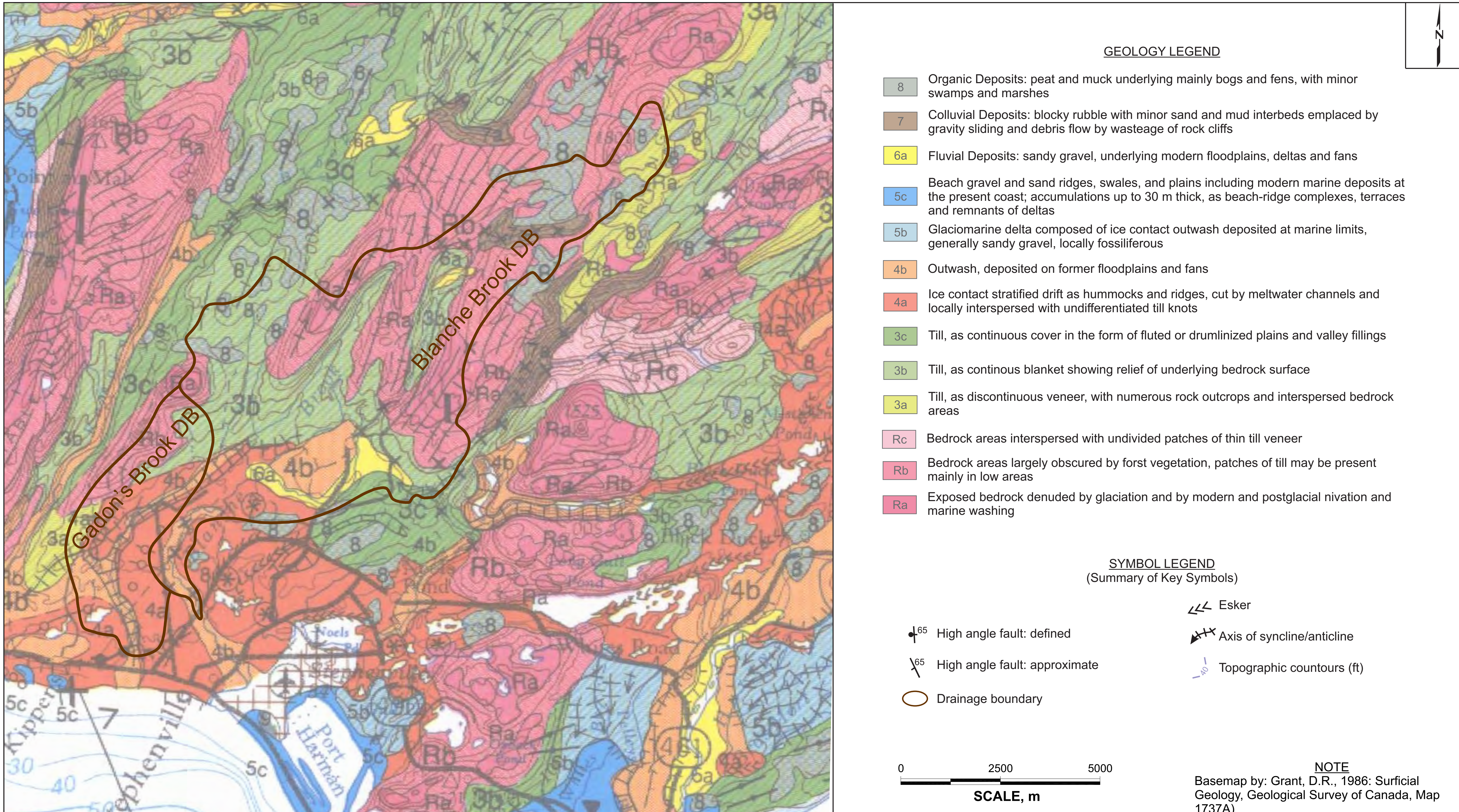



Figure 2.2 Surficial geology in the Gadon's Brook and Blanche Brook drainage basin.

| | |
|---|---|
| Project No. 555 | Document Reference As Shown |
| Location Stephenville, NL | Date July 2021 |

FFC

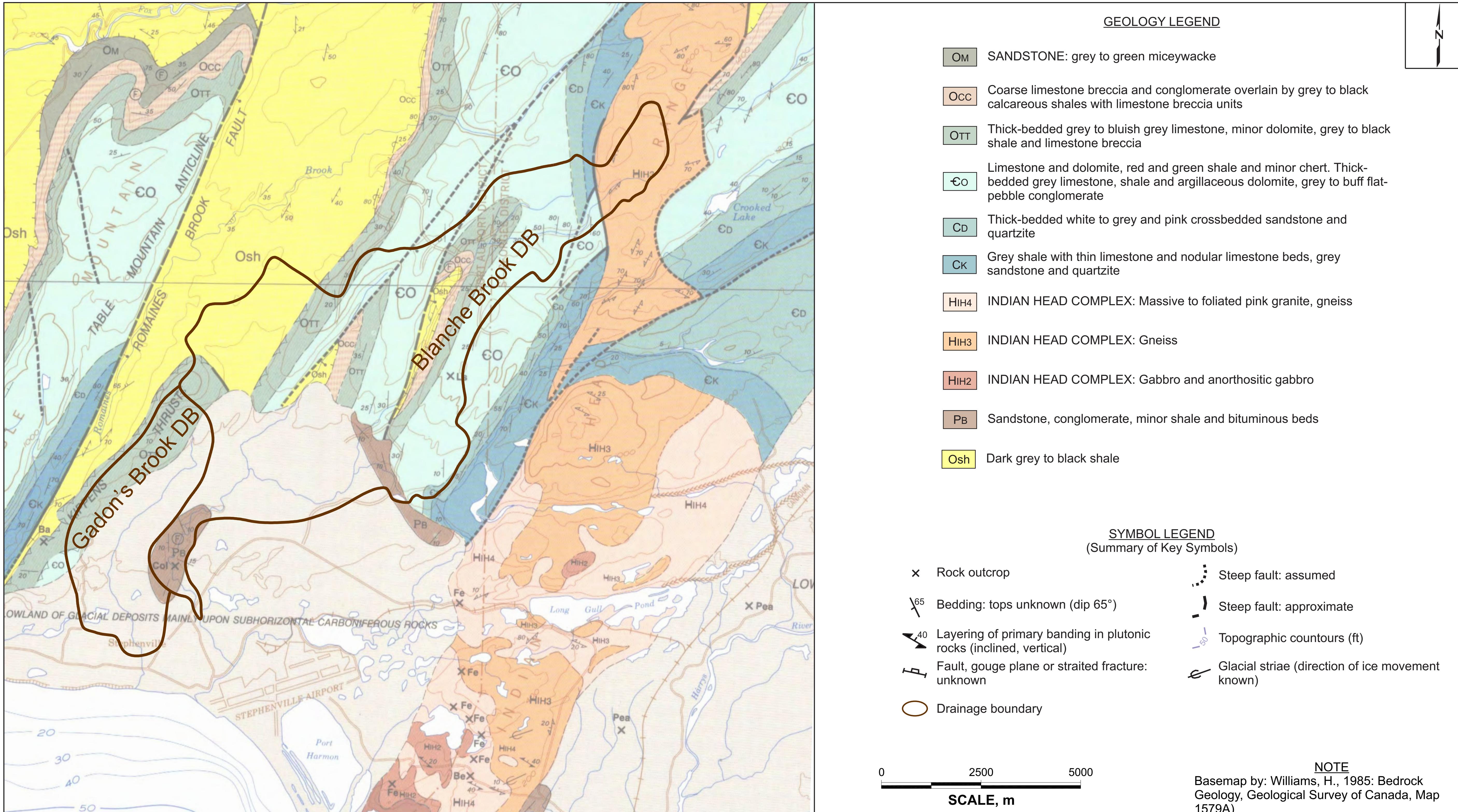
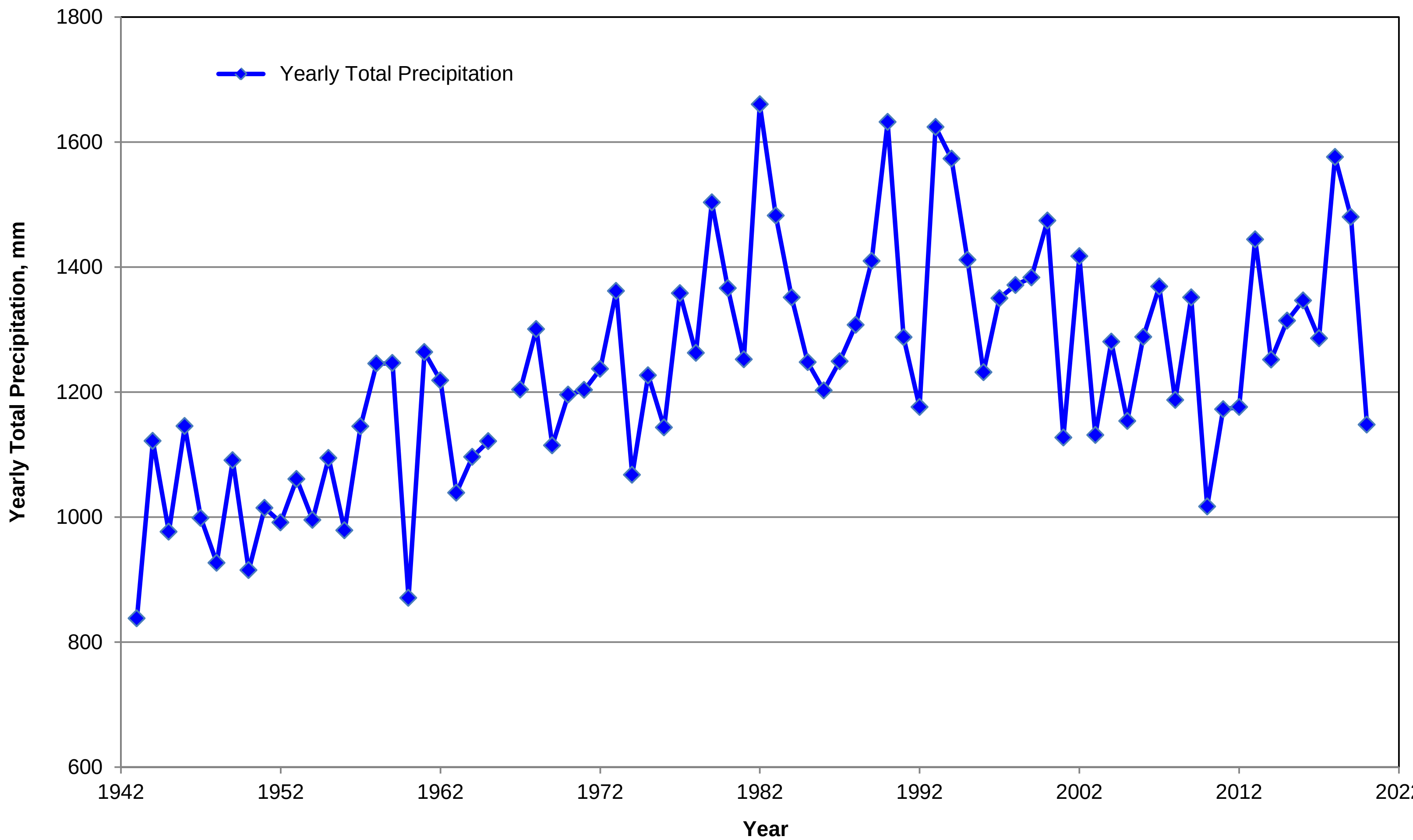



Figure 2.3 Bedrock geology of the Gadon's Brook and Blanche Brook drainage basins.

| Project No. | Scale |
|---|---|
| 555 | As Shown |
| Location | Date |
| Stephenville, NL | July 2021 |

FFC




| | | | |
|---|---|---|---|
| Figure 2.4 | Yearly total precipitation from 1942 to 2020 at the *Stephenville A* climate station (EC, 2021). | Project No. 555 | Document Reference FFC-NL-555-004 |
| | | Location Stephenville, NL | Date June 2021 |

FFC

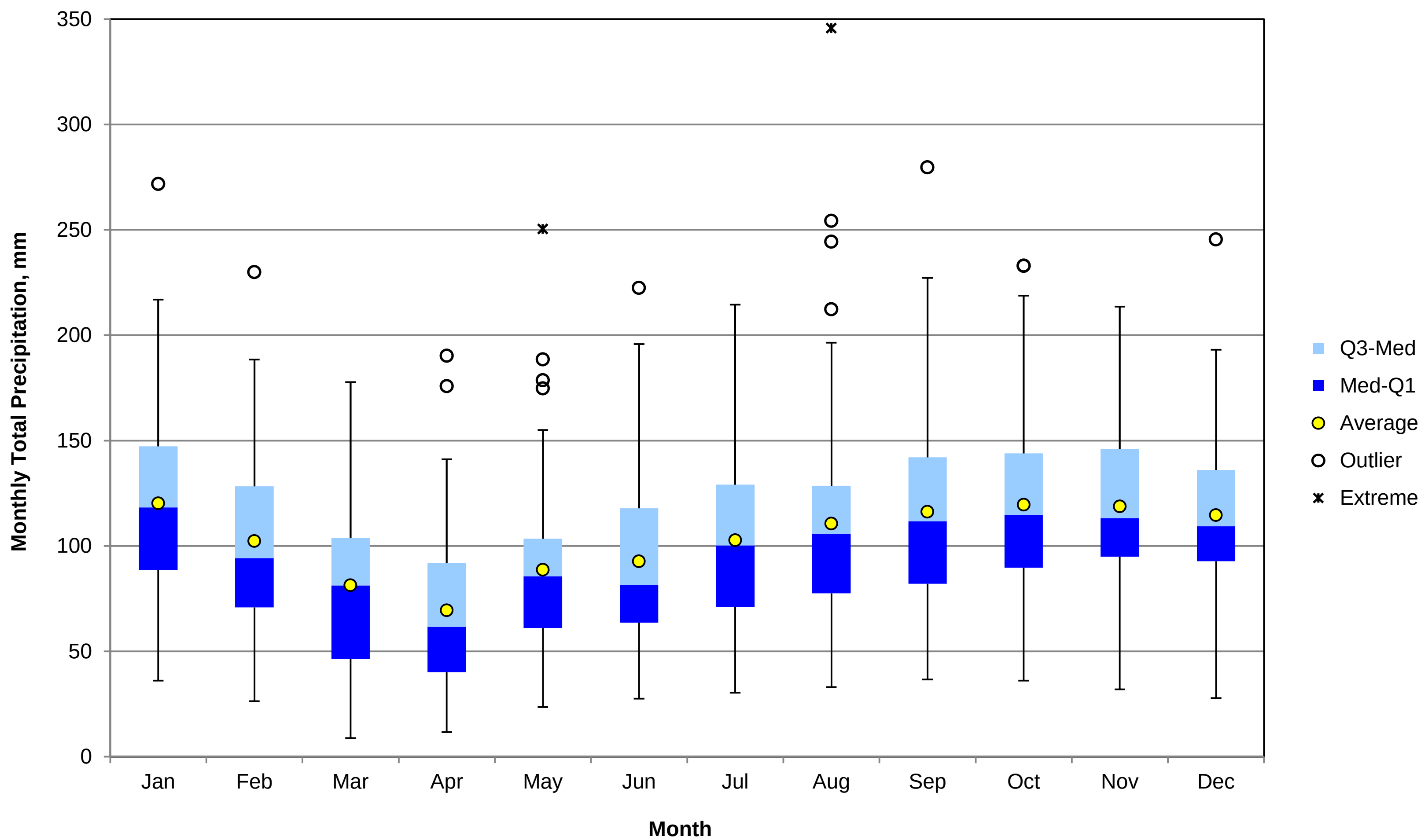



Figure 2.5 Monthly total precipitation from 1942 to 2021 at the *Stephenville A* climate station (EC, 2021).

| Project No. | Document Reference |
|---|---|
| 555 | FFC-NL-555-004 |
| Location | Date |
| Stephenville, NL | June 2021 |

FFC

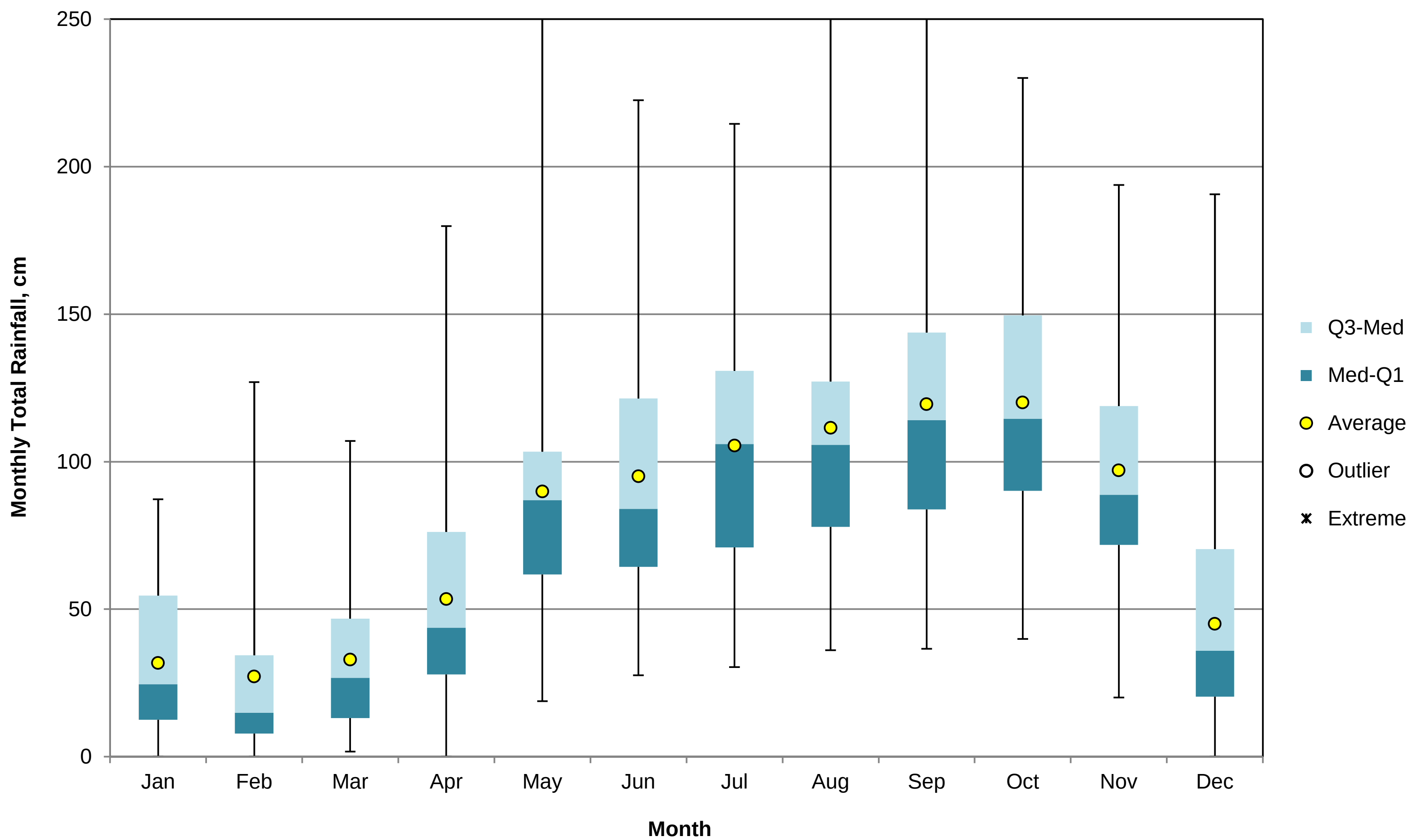



Figure 2.6 Monthly total rainfall from 1942 to 2021 at the *Stephenville A* climate station (EC, 2021).

| Project No. | Document Reference |
|---|---|
| 555 | FFC-NL-555-004 |
| Location | Date |
| Stephenville, NL | June 2021 |

FFC

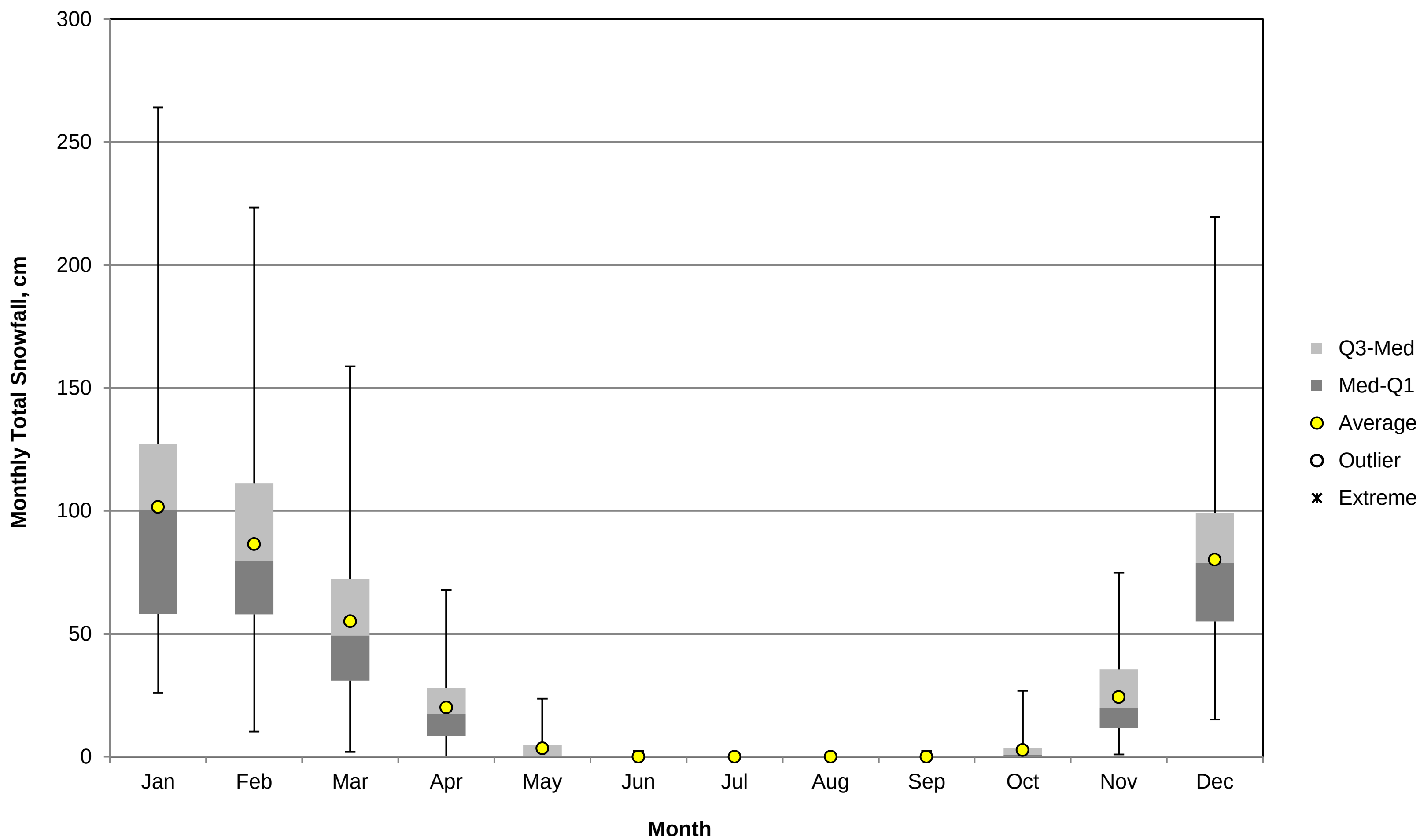



Figure 2.7 Monthly total snow fall from 1942 to 2021 at the *Stephenville A* climate station (EC, 2021).

| Project No. | Document Reference |
|---|---|
| 555 | FFC-NL-555-004 |
| Location | Date |
| Stephenville, NL | June 2021 |

FFC

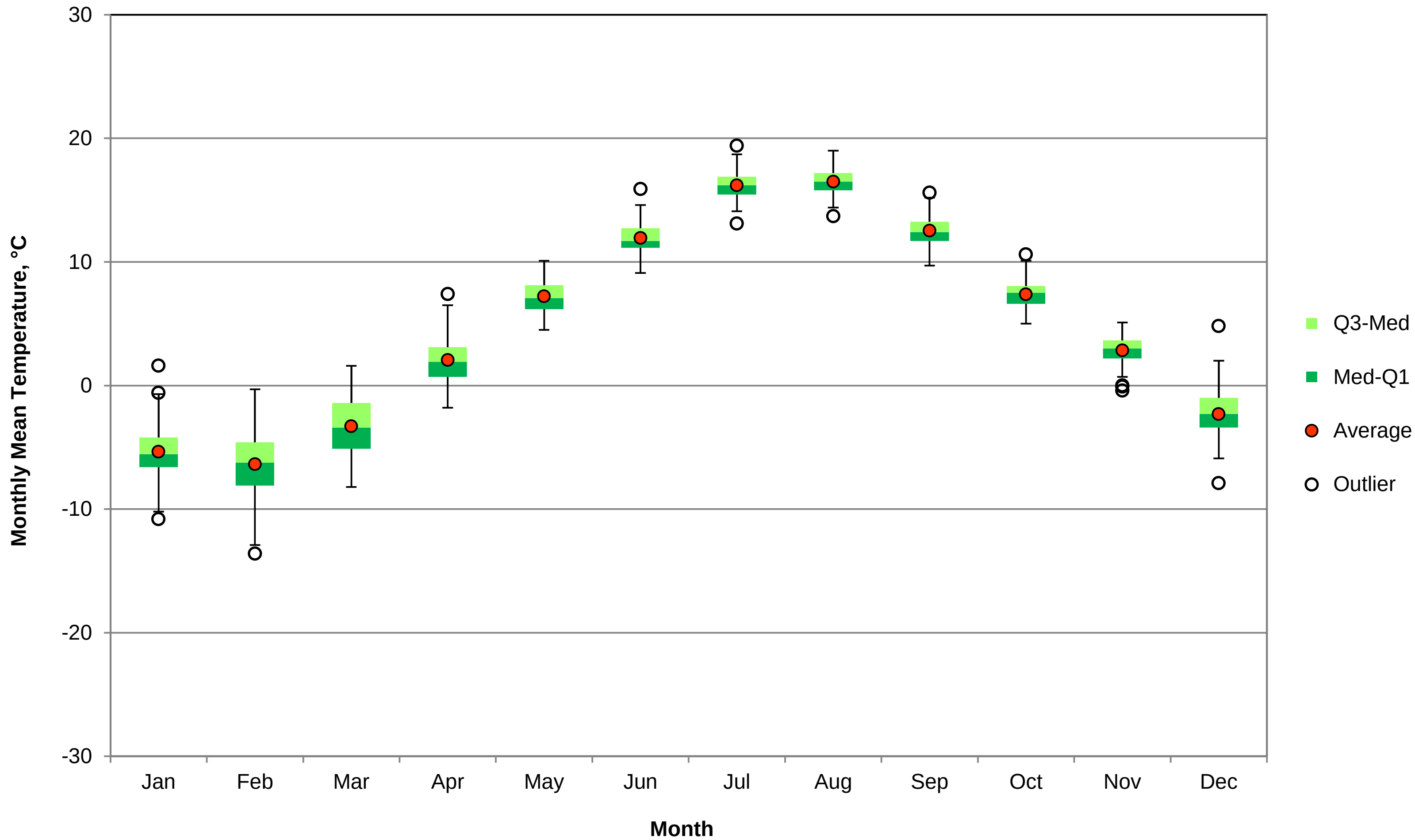



Figure 2.8 Monthly mean temperature from 1942 to 2021 at the *Stephenville A* climate station (EC, 2021).

| | |
|---|---|
| Project No. 555 | Document Reference FFC-NL-555-004 |
| Location Stephenville, NL | Date June 2021 |

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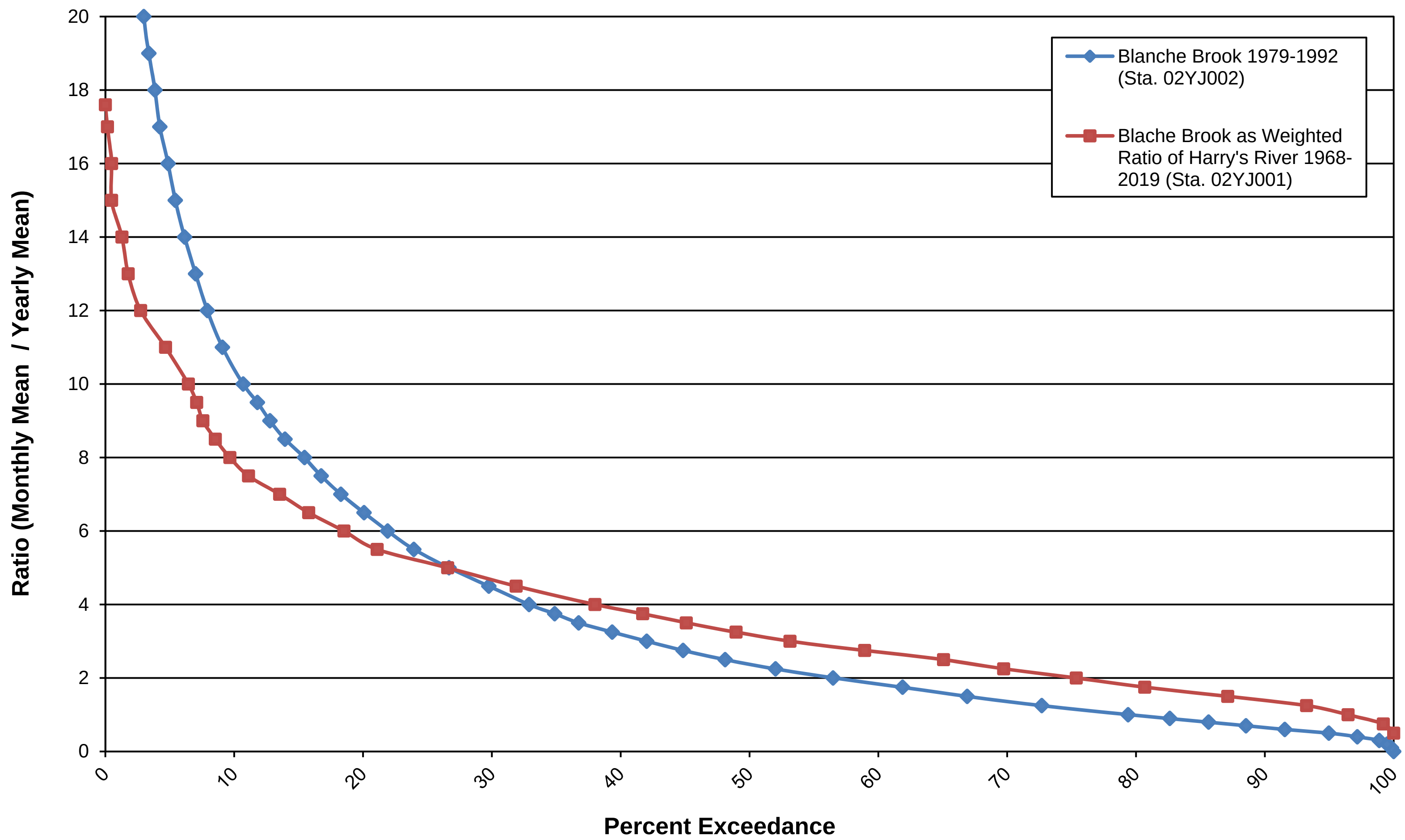



| | | |
|---|---|---|
| Figure 2.9 Flow duration curve of observed flow values from Blanche Brook (Sta. 02YJ002) and calculated values from Harry's River (02YJ001) | Project No. 555 | Document Reference FFC-NL-555-004 |
| | Location Stephenville, NL | Date June 2021 |

FFC

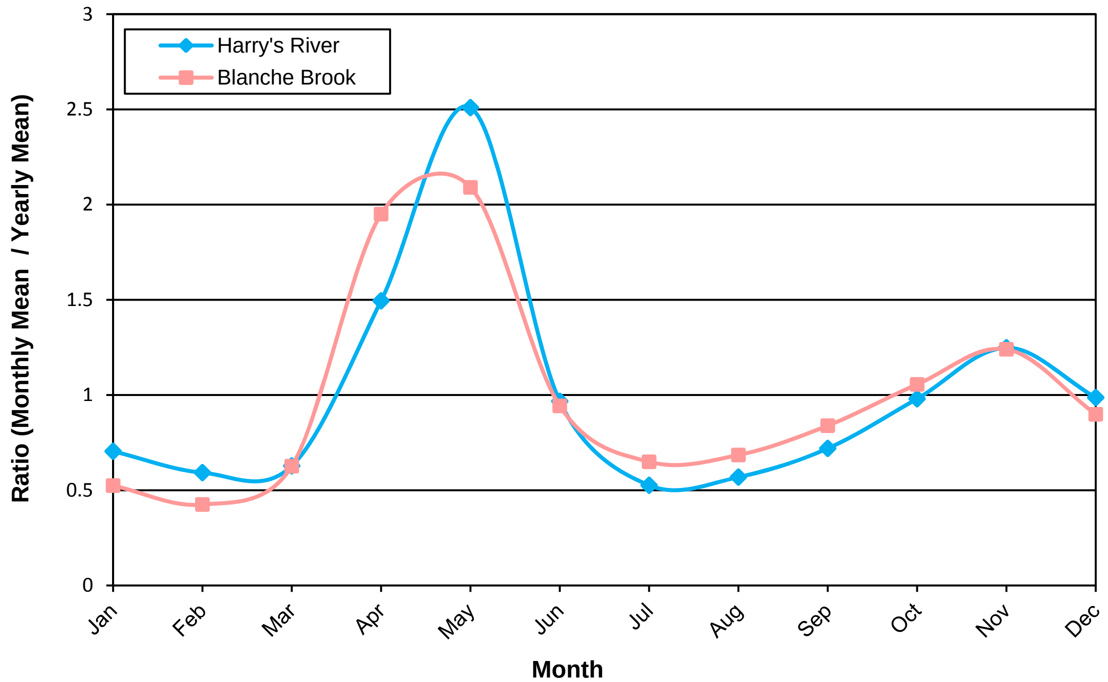



Figure 2.10 Ratio of mean monthly flow to mean yearly flow for Blanche Brook (Sta. 02YJ002) and Harry's River (Sta. 02YJ001)

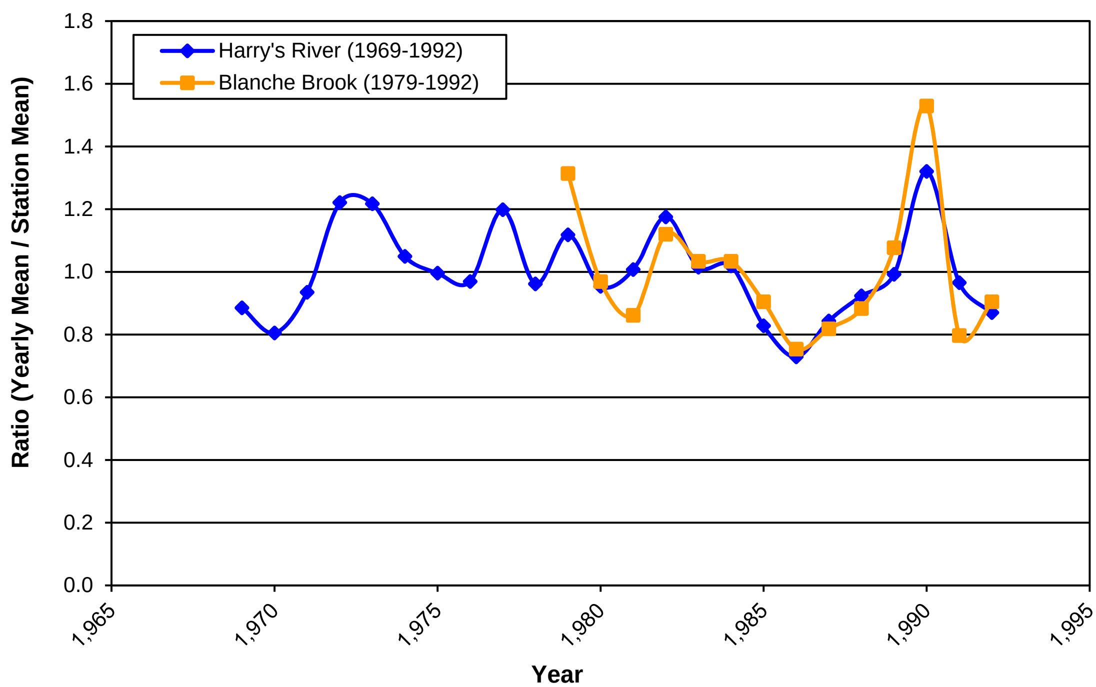



Figure 2.11 Ratio of mean yearly flow to the stations mean flow for Blanche Brook (Sta. 02YJ002) and Harry's River (Sta. 02YJ001)

# 3.0 HYDROGEOLOGICAL PROPERTIES

As shown by the production and observation well logs and confirmed by the aquifer tests, there are two main aquifer systems in the Blanche Brook drainage basin, the overburden aquifer and the bedrock aquifer, both of which supply water to the existing well field. The existing Stephenville production well that produces primarily from the bedrock aquifer is Well 1. All of the remaining eight production wells produce primarily from the overburden and shallow bedrock aquifer system. The four production wells that service the Town of Kippens are all considered to be completed in the same bedrock aquifer that supplies part of the water supply for the Town of Stephenville.

The geological logs for each of the Stephenville water supply wells and the original site investigation wells are provided in **Appendix A**.

## 3.1 Hydrogeological Properties and Well Yield

The overall hydrogeological properties are summarized in this section and expanded in the following numerical model chapter. **Table 3.1** summarizes the site conditions including the original depth to water and the aquifer test data including the calculated transmissivity and storativity values for each well.

## 3.2 Existing Well Field Performance

The Town has been operating its groundwater wells for approximately 20 years. During this period, a number of the water wells have experienced significantly reduced well yields and reduced specific capacity. After subsequent redevelopment by simultaneous surge blocking and air-lifting, the original specific capacity of most of the wells was restored. However, the static water level in Well 1, the high capacity deep bedrock well, was substantially lower with a corresponding reduction in well yield.

**Table 3.2** provides, for comparison, reference data showing the original specific capacity for each well, the maximum well yield during the original aquifer test, the original recommended short duration maximum pumping rates, and the revised recommended maximum pumping rates based on the initial aquifer performance along with the pumping rates that were recorded on January 28, 2021. Based on the 2021 pumping rates, only one well, Well 4, is currently producing above its recommended pumping rate.

Since the static water level in Well 1 was significantly lower, when last measured, than the original static water level in the bedrock aquifer, it is reasonable to assume that at the higher pumping rates, the bedrock aquifer was being over-exploited.

The static water levels in Well 7, Well 8, Well 9 and Well 10, are reported to generally return to approximately the same level as the original measured static water levels, after the pump has been shut down for a 24- to 48-hour period, with adjustments for well interference from nearby operating wells. If current measurements of static water levels in those four wells confirm that the static water levels in the aquifer are consistent with the original static water levels, and there is no evidence of bacterial growth on the well screens, then it is reasonable to assume that the well yield and specific capacity of those four wells can be restored by aggressive well re-development using simultaneous surge blocking and air-lifting followed by measurement of the specific capacity of each well to confirm that the re-development work has achieved the desired outcome.

For the other three water supply wells, Well 2, Well 5, and Well 6, the current static water levels need to be measured to determine if the overburden/shallow bedrock aquifer in which those wells are completed is being over exploited. If the static water level is acceptable, and no bacterial growth is present on the well screens, then the specific capacity and yield of those three wells can also be restored by an aggressive program of well re-development.

Table 3.1 Summary of site/field conditions and data from aquifer tests that was conducted in each production well in 1999 (Fracflow, 1999b).

| | | **Site Condition** | | | | **Aquifer Test** | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Well ID | | Static Water Depth | Ground Surface Elev. | Water Elev. | Well Depth | Pumping Rate | | Draw-down | Max. Yield | Field[1] Transmi-ssivity *T* | Stora-tivity[1] *S* |
| Town | Drilling | m | m | m | m | L/min | GPM | m | L/min | m2/s | -- |
| Well 1 | PTW1 | 5.61 | 70.87 | 65.26 | 45.1 | 1445 | 381.7 | 6.39 | 3706 | 2.00E-03 | 7.50E-04 |
| Well 2 | PTW5 | 0.77 | 65.48 | 64.71 | 20.7 | 1175 | 310.4 | 1.45 | 3839 | 2.50E-02 | 5.00E-02 |
| Well 3 | PTW7 | 1.43 | 66.77 | 65.34 | 17 | 1552 | 410.0 | 3.96 | 2391 | 4.20E-02 | 2.00E-03 |
| Well 4 | PTW6 | 1.86 | 67.23 | 65.37 | 18.8 | 1180 | 311.7 | 1.31 | 7168 | 2.20E-02 | 1.00E-01 |
| Well 5 | PTW2 | 8.58 | 73.09 | 64.51 | 34.7 | 500 | 132.1 | 7.59 | 1008 | 1.60E-03 | 8.00E-03 |
| Well 6 | PTW8 | 7.96 | 73.05 | 65.09 | 24 | 320 | 84.5 | 4.63 | 421 | 4.50E-03 | 3.00E-06 |
| Well 7 | PTW9 | 0.3 | 80.3 | 80 | 32.2 | 1805 | 476.8 | 15.85 | 2330 | 7.80E-03 | 5.00E-04 |
| Well 8 | PTW10 | -0.91 | 79.97 | 80.88 | 33 | 1800 | 475.5 | 14.49 | 2699 | 7.50E-03 | 1.00E-04 |
| Well 9 | PTW11 | 0.24 | 84 | 83.76 | 24 | 1235 | 326.3 | 12.64 | 922 | 2.30E-03 | 5.00E-04 |
| Well 10 | PTW12 | 0.52 | 86 | 85.48 | 26 | 927 | 244.9 | 11.28 | 790 | 2.80E-03 | 5.00E-04 |

1 Transmissivity and storativity values were extimated using average values extracted from the previous project based on a series of aquifer tests.

Table 3.2 Summary of specific capacity, maximum yield and recommended production rates from the previous field tests with the current pumping rates in the nine production wells at the Stephenville well field.

| Production Well | Specific Capacity<br>L/min/m | Maximum Yield<br>L/min | Recommended Maximum Pumping Rate - 1999<br>L/min | Recommended Maximum Pumping Rate - 2004<br>L/min | Flow Rate on Jan. 28, 2021<br>L/min |
|---|---|---|---|---|---|
| Well No. 1 (PTW-1) | 226 | 3706 | 2,000 to 2,600 | 1,900 | 1,302 |
| Well No. 2 (PTW-5) | 812 | 3839 | 1350 | 800 | 180 |
| Well No. 3 (PTW-7) | 394 | 2391 | 1350 | na | na |
| Well No. 4 (PTW-6) | 881 | 7168 | 1350 | 1,200 | 1,396 |
| Well No. 5 (PTW-2) | 66 | 1008 | 778 | 700 | 242 |
| Well No. 6 (PTW-8) | 68 | 421 | 310 | -- | 83 |
| Well No. 7 (PTW-9) | 114 | 2330 | 1350 | 1,400 | 1,030 |
| Well No. 8 (PTW-10) | 124 | 2699 | 1300 | 1,500 | 813 |
| Well No. 9 (PTW-11) | 98 | 922 | 655 | 650 | 307 |
| Well No. 10 (PTW-12) | 82 | 790 | 600 | 570 | 241 |

# 4.0 NUMERICAL MODELING

As noted, the original field program showed that the adjoining towns of Stephenville and Kippens share a common fractured bedrock aquifer that supplies part of the municipal water supply for Stephenville and all of the water supply for the Town of Kippens. The balance of the water supply for the Town of Stephenville is supplied by screened water wells that are completed in the overburden and the shallow bedrock. Approximately four of the Stephenville wells obtain part of their well yield through induced infiltration from Blanche Brook with potential impacts on baseflow in the brook during periods of low rainfall.

The original MODFLOW model that was constructed in 1999 was used to give a general map of the groundwater flow patterns in the drainage basin under both natural conditions and under well field pumping conditions to assess the impact and capture areas produced by long-term drawdowns. Since the MODFLOW model is a finite difference model that uses a regular grid mesh, for a large area with distributed wells such as exists for the Stephenville site, it is very difficult to develop a detailed mesh, or very small grid, around each well, using a finite difference code, in order to simulate travel times, identify exclusion zones, map groundwater flow pathways, refine watershed boundaries and quantify impacts to the baseflow in Blanche Brook from induced infiltration.

This revised model was constructed using FEFLOW (Finite Element subsurface Flow System). FEFLOW (Diersch, H. -J. G., 2005) is an advanced, finite element code that is used to model groundwater flow and transport in both porous media and fractured-bedrock systems. The transport portion of the FEFLOW code allows the user to track the movement of particles, or tracers, along discrete flow lines to map the direction of groundwater movement and travel time of particles in the water such as both conservative (Chloride) as well as non-conservative (organics) ions between points of interest. Unlike the finite difference code, the finite element approach is more suited to simulate multiple production and observation wells because the mesh around the wells can be refined locally, to produce a significantly smaller number of grids with shorter simulation times than the finite difference code, which allows one to map the local drawdowns and small capture areas.

In addition, in this revised model, the new extended model boundaries (**Figure 4.1**) were selected to capture the more extensive drainage basin boundaries and to enable the assessment of other potential water supply areas. The elevation data were also updated using the latest data available in a form of an ortho-rectified image and its associated digital elevation data. This allowed the model to capture more detailed local and regional groundwater flow systems, such as recharge and discharge areas, and to identify potential future water supply areas in case the Town decides to construct additional water supply wells to supplement the existing production wells. The well field for the Town of Kippens was also included in the updated model area to assess the impact of pumping from the Town of Kippens' well field on the Town of Stephenville's well field. Based on the calibrated flow model under the current pumping condition, the impact of potential contaminates such as the leachate from the abandoned and active landfill was re-evaluated using a transport model by calculating travel time from the impacted sites to the pumping wells. In addition, the risk from small point source releases of waste water from residential developments was assessed.

This model was designed to be used primarily to evaluate the sustainability of the flow from Blanche Brook to the well field and the impact of lowering the water table and extracting more water using the existing well field and the additional withdrawal rates that would be imposed by one or more new production wells. A simplified conceptual hydrogeological model of the extended Stephenville, Blanche Brook, Cold Brook and Gadon’s Brook drainage basins was developed for use in modelling the groundwater flow field and the potential impacts of changes in the current water supply withdrawal rates on the regional and local groundwater flow system. The conceptual hydrogeological model was constructed based on a combination of the available hydrogeological data and informed judgement. Every attempt was made to incorporate field measurements collected at the site during this study and previous studies. However, assumptions were still required in most locations since the data coverage is sparse in some areas.

## 4.1 Model Construction

The overall study area or model domain that was selected for the extended groundwater flow simulation for the Town of Stephenville well field covers approximately 76.5 km$^2$ (**Figure 4.1**) of the overall drainage basins. The study area was divided into a number of discrete nodes and elements for which all of the hydrogeological parameters were assigned for model simulations. The nodal points define the corners of a series of triangular/polygonal elements creating what is referred to as the surface mesh (**Figures 4.2, 4.3** and **4.4**). The nodal points have been specified across the surface of the study area at strategic locations such as river/stream, wells, ponds, shore lines, etc. The surface mesh was constructed with a total of 23,829 nodal points and 47,037 elements. The starting mesh was created using coarser elements in which the surface width/length of each element varied between 70 m and 100 m. These elements were then refined into smaller elements to allow the properties and boundary conditions of the areas of interests (**Figure 4.2)** to be represented.

In order to simulate drawdowns around a pumping well, the corresponding node for each well and its surrounding elements were further refined gradually until the elements forming the well and the area around each well bore had dimensions that were of the same order of magnitude as the diameter of the pumping well. An example of the well refinement is shown in **Figure 4.3** and **Figure 4.4**. The smallest element width at each well centre was 0.75 m in length. In the vertical direction, the domain was divided into 16 layers (17 slices) to represent the different layers of overburden material and the underling bedrock with a combined thickness that ranged from 500 m at the shoreline to 898 m at the highest elevation point at the top of the drainage basin. This produced a 3D model with 405,093 nodes and 752,592 elements. The grid was also smoothed to remove any obtuse angles within each element. Without grid smoothing, abrupt changes in grid spacing could have caused non-convergence of the numerical solution and a large mass balance error in the volume of water within the modelled area. **Figure 4.5** shows a cross-section of the model which illustrates the 3D model geometry with elevation data.

The surface/ground elevation of the model domain (Slice 1) was constructed based on a digital elevation model (DEM) data set derived from a set of ortho-rectified images that were obtained from the Department of Fisheries and Land Resources. The surface elevation was assigned to each surface node (nodal points of elements on Slice 1) in the model using one of the FEFLOW's interpolation routines (i.e., inverse distance-squared method). Due to the differences between the data interval and element size, some level of smoothing of the elevation data is produced during the ground surface interpolation process.

The stratigraphy used in the model was based in part on the borehole data base from the previous reports (Fracflow, 1999a, 1999c and 2004), Fracflow's internal data files and geological reports and maps from government's public database (Williams, 1985 and Grant, 1986). The model stratigraphy consisted of an upper till and/or organic/bog layer, sand layer with thin sporadic/scattered silt/clay layers, a thick sand layer, and the underlying bedrock. In order to accommodate the variable properties of the geological layers, the thickness of each type of overburden and bedrock that was assigned to each of the various model layers was estimated based on the available borehole information. However, the borehole database is sparse and the depth to the bedrock (boundary between overburden and bedrock) has not been measured in some of the existing production and monitoring wells and in the overall drainage basin area. In those locations, the thickness was estimated based on the surficial and bedrock geology reports and maps. There was no evidence that significant confining layer (impermeable or semipermeable layer such as clay) exists throughout the well field area. Based on the well logs, many locations had clay, silty clay, or silty sand layers with variable depths of those layers below the ground surface along with variable layer thickness. To accommodate the variable thickness of the geological layers, several 5 m thick layers were assigned at the top of the model. Then the layer thickness was increased with depth. At the bottom of the model, the bedrock unit was assigned using several layers to represent the general tendency of decreasing hydraulic conductivity values with increasing depth. The thickness and depth of each layer and its geological description are presented in **Table 4.1**.

## 4.2 Model Input

The numerical flow model is controlled by a number of different input parameters, which are assigned to each node or each element to simulate the hydrogeological features of the model domain. These parameters include (1) the hydraulic boundary conditions with known hydraulic heads such as streams, ponds, lakes and ocean, and measured hydraulic head data from monitoring or pumping wells, (2) hydraulic conductivity values for the transmissive properties of representative layers obtained from aquifer tests on production wells, (3) the amount of infiltration or recharge to the different areas of the model domain, and (4) any known groundwater sources and sinks such as injection or pumping wells and corresponding water quantities for domestic or industry usages. In general the water storage properties of the various layers must also be specified. However, for steady-state simulations, these parameters are not required.

The hydraulic boundary conditions were assigned to the major surface water bodies in the interior of the drainage basin. The streams/brooks were specified as constant head boundaries where this condition was supported by the water table elevations. The values of the constant head cells that were assigned along the brooks corresponded to an elevation that was approximately 0.5 m to 1.5 m lower than the ground surface elevation for each cell that was used to define the brooks. The constant head boundaries were only specified for the top slice of the model layer. For most cases, it was implicitly assumed that the high permeability bed of the streams/brooks was no deeper than the top layer (i.e., 5 m) of the model. Considering that streams/brooks in this area are generally not deeply incised features, this was considered a reasonable assumption.

There are two major ponds near the Town's well field, Beaver Pond and Ned's Pond, that were also used as constant head boundary with the pond's water surface elevations assigned as the hydraulic heads. For the south side of the model boundary at the St. George's Bay, a constant head of 1 m above sea level was assigned along the shoreline to reflect average tide changes. This boundary condition was applied to all layers of the model along the outer shoreline except the bottom layer. In coastal environments, groundwater flow is generally directed upwards as the sea boundary is approached and the model boundary is assigned a no-flow boundary condition. This assignment is based on the expected response to the density contrast between seawater and the overlying fresh water. In this situation the seawater effectively acts as an impermeable boundary to flow unless the horizontal hydraulic conductivities are much higher than the vertical hydraulic conductivities. However, for this site the layered nature of the granular aquifer results in significant flow of ground water into the ocean from each layer. **Figure 4.6** shows the selected streams/brooks and ponds that were represented as fixed hydraulic boundary conditions. For locations where the surface water could not be used to define boundary conditions, the perimeters of the drainage basins or hydraulic boundaries were defined by assuming that groundwater divides coincide with topographic divides.

Hydraulic transmissivity(T)/conductivity (K) values assigned to the model layers were estimated based on compiled hydraulic conductivity data from a series of aquifer tests conducted at the project site (Fracflow, 1999a, 1999b and 2004). The initial conductivity values that were used in the model are tabulated in **Table 4.2**. The conductivity values were then adjusted during the model calibration process to accommodate the variable thickness and spatial variability of the geological units. Note that the vertical hydraulic conductivities ($K_v$) are approximately a half to one magnitude order less than the horizontal hydraulic conductivities ($K_h$). In addition, the conductivity assignment was initiated with an assumption that the conductivity values were isotropic in the horizontal plane. However, based on the calibration results, the conductivity values were updated locally to accommodate the anisotropic nature of the fractured rock system. **Figure 4.7** is a cross-section of the model with illustrates the 3D model geometry with a cut-out section showing the distribution of the assigned hydraulic conductivity values within the model.

The model domain was divided into four zones for recharge assignment based on the surficial geology map of Stephenville-Port Aux Basques (Grant, 1986). The pattern of recharge zones is shown in **Figure 4.8**. Zone 1 was mainly covered with generally 5 to 30 m thick sandy and silty overburden over bedrock; Zone 2 was assigned to the highlands or higher elevation area with 2 to10 m thick overburden; Zone 3 was mainly over the area with direct bedrock exposure in the higher elevations of the drainage basin; and Zone 4 represents the areas of bog or marsh that are visible on aerial photographs. The groundwater recharge rates for the different parts of the model area were estimated as a percentage of the total precipitation recorded in the historical climate data based on assumptions regarding evapotranspiration, estimated runoff from gauged drainage basins and estimates of snow losses due to sublimation.

Groundwater recharge rates for this area are typically between 20% and 30% of total precipitation in good ground conditions and the full range of recharge rates can be expected to occur locally over most of the Blanche Brook and Beaver Pond drainage basin area which was assigned as Recharge Zone 1. Zone 2 covers the other areas in the drainage basin that have forest cover and granular overburden that varies in thickness with a range of depths to the water table. In these sections of the drainage basin, groundwater recharge was assumed to contribute recharge primarily to the granular aquifer. Normally, the lowest recharge rates will be associated with exposed bedrock (Zone 3) because the majority of the precipitation runs off to streams and lower ground surface. Evaporative and evapotranspiration losses are expected to be high for part of the year. Bog/marsh areas within the model domain were outlined based on the ortho-rectified image and the surficial geological map and assigned as Zone 4. Since the thick marsh layer is fully saturated due to the high porosity of the bog or peat material and the perched nature of the upper water table, the water in this upper layer is available to recharge the granular aquifer system below the marsh areas during the entire year because they act as a sponge that hold the rain and snow meltwater and gradually releases the water to the underlying aquifer. Therefore, an enhanced recharge value was assigned on these areas. It was assumed that the known bog/marsh areas are underlain by some degree of semipermeable layer, so a lower conductivity value was assigned to these areas.

## 4.3 Model Calibration

The main purpose of calibration processes for a model is to confirm that the model can reproduce the measured hydraulic head data and areas with known water levels at the surface with corresponding sinks/sources of water throughout the model domain. A typical calibration process starts with a set of predetermined hydraulic properties/parameters, such as head boundary conditions, hydraulic conductivity values, and recharge rates distributed across the model area. The properties/parameters are then updated based on the simulated head distribution until an acceptable match between the measured hydraulic heads and the model calculated heads is obtained. In general, it is difficult to achieve an acceptable calibration result of a model by using head measurements alone. The lack of an acceptable calibration based on the head distribution alone can be demonstrated by Darcy’s law that shows changes in either hydraulic conductivity or recharge will produce the same effect on the head distribution. Therefore, flow measurements and corresponding head changes from known sources of water extraction from wells and/or injection tests such as an aquifer test, are required as additional input parameters to reduce any uncertainty and improve the calibration results. However, it should be noted that although the successful calibration of a model is an essential condition for a unique flow system, it is not always a sufficient condition. The goal of the calibration step is to ensure that the 3D model is a reasonable approximation of the natural groundwater flow system in the drainage basin.

The calibration process started with a model under a non-pumping condition to simulate the groundwater flow system in the drainage basin prior to any stresses being applied to the corresponding aquifer system. For calibration of the non-pumping condition, the hydraulic head data were extracted from the aquifer tests conducted in ten production wells between 1997 and 1999 which was prior to the operation of the Town's water supply system. A location map of the production and monitoring wells used in this model calibration process is presented in **Figure 4.9**. In order to use a hydraulic head data set for a numerical simulation, the hydraulic data should be collected under the same weather and hydraulic conditions, since the hydraulic head distribution varies with changes in annual precipitation levels, the season of the year, and the water usage from the aquifers. However, the aquifer tests from the previous projects were carried out over an extended period. Aquifer tests in the ten Stephenville wells were conducted in December 1997 and January to April 1999 and in the three Kippens wells in August 1997 and November to December, 1999. Therefore, one has to recognize that the error bar in matching measured and computed hydraulic heads will reflect the range of groundwater recharge conditions. It should also be noted that the hydraulic head data were measured during both winter and spring months when the variation in recharge rates are high due to snow meltwater.

There are at least 30 wells (10 production wells and 20 monitoring wells) in the Stephenville well field area and four production wells in the Kippens well field area. **Figure 4.9** shows the overall distribution of the wells within the model domain and **Figure 4.10** shows the well locations and well ID and the corresponding hydraulic heads as presented in **Table 4.3**. The three Kippens production wells were included in the current model to provide a boundary condition for the model and to assess the impact of the long-term pumping of the four production wells on the deep bedrock well that are operated by the Town of Stephenville. For some of the model simulations, only three of the production wells in the Kippens well field were included.

A typical trial-and-error approach was used to obtain a steady-state calibration for the flow model simulation. Starting with the initial parameters that were determined based on the historic climate data and the field measurements from the aquifer test data, both recharge and hydraulic conductivity values were adjusted until a reasonable range was reached between the measured and calculated heads. Selected scenarios of the recharge values for the four zones are presented in **Table 4.4** including the recharge values assigned for the final simulation that ranged from 7% to 28%. The final conductivity values that were assigned to the model are presented in **Table 4.1**. The initial conductivity input values were gradually decreased by the same proportion for each layer to match the calculated head data to the measured hydraulic heads. Once the overall head differences (errors) were reduced to an acceptable range, the hydraulic conductivity values for each layer were further adjusted under pumping conditions using the drawdown data from the aquifer tests. Steady-state simulations were used to compare the drawdowns that were measured in the monitoring wells to the calculated hydraulic head changes at the same elevations/depths as the well screens were installed on the corresponding monitoring wells. Then the conductivity values on the corresponding layer(s) were adjusted to match the drawdowns and the pattern/extent of the drawdown cone.

**Figure 4.11** is a plot of measured hydraulic heads in the available monitoring wells versus the hydraulic heads that were calculated by the 3D model at the corresponding wells. For a perfect match between measured and compute hydraulic heads, the data points would plot on a 45-degree line which is shown as a solid line in **Figure 4.11**. For the perfect fit, this line would have a regression $R^2$ fit of 1.0. For the actual calibration, the dashed lines on either side of the solid line indicate a +/- difference of 5.0 m between the measured and computed hydraulic head values. For data points that plot below the solid line, the measured (field) heads are higher than the 3D model computed heads and for data points that plot above the solid line, the computed heads are higher than the measured heads. The $R^2$ term indicates the degree of fit between the measured and computed heads. Based on the high degree of agreement ($R^2 = 0.94$) between the measured and computed hydraulic heads, the model was considered to be calibrated as an acceptable approximation of the real groundwater flow system in the model area and this calibrated model was then used for this series of groundwater flow system and aquifer simulations. The water table map from the calibrated model under steady-state non-pumping condition is shown in **Figure 4.12**.

For the final step of the calibration process, the calibrated model was used to simulate the current pumping condition of the Town’s well field. A well record database for the last five years (2016 to 2021) of operation was provided by the Town of Stephenville. To simulate a long-term pumping under steady-state condition, an average pumping rate from the five-year well records was assigned to each of the nine production wells. **Table 4.5** presents the summary of the five-year well records for the nine production wells, showing five-year average flowrates, monthly average flowrates for January 2021, daily average flowrates on January 28, 2021, and model input flowrates. The hydraulic heads were measured on January 2021 in the available wells inside the Town’s well field. However, among the 20 monitoring wells used in 1999, only nine wells were identified in January 2021 (**Table 4.6**). The differences in depth to water were compared to the drawdowns from the long-term pumping simulation. **Figure 4.13** shows a plot of the measured hydraulic heads in the available monitoring wells in January 2021 versus the computed hydraulic heads under pumping conditions. As shown in **Table 4.6** and **Figure 4.13**, larges differences were detected in LF1-4 and PTW4.

The model calibration simulations also helped to identify apparent variations in the overburden and bedrock aquifers. For example, the measured water levels in PTW4, completed in bedrock, at the north and northeast end of Beaver Pond had much higher drawdown (5.02 m) than was computed by the 3D model for pumping in Well 1. Also, PTW4 which is further away from Well 1, the pumping well, had a much larger drawdown than Well 5 which had a drawdown of only 0.18 m for the same pumping rate in Well 1. This large drawdown in PTW4 indicates that the high permeability zone in the bedrock aquifer does not extend much beyond the northeast end of Beaver Pond and that the bedrock aquifer is being over-exploited and that additional water supply wells cannot be constructed in the bedrock northeast of Beaver Pond.

Similarly, when Well 10 was being pumped, the drawdown in Well 9 was measured at 3 m but the 3D model only shows about 0.2 m of drawdown for long term pumping of Well 10. This difference in measured and computed drawdowns indicates that the overburden/bedrock aquifer north of Well 9 has a zone of lower permeability or that the permeable zone is somewhat discontinuous.

Based on this difference in computed and measured drawdowns in PTW4, as noted additional water supply wells cannot be constructed in the bedrock aquifer in any area northeast of Beaver Pond. Also, the difference between the measured and computed drawdowns in Well 9 indicates that there is significant variability in aquifer properties in the area of Well 9 and Well 10. Since the area that is immediately north of these two production wells is the primary site for one or more new production wells, along the Cold Brook tributary, potential well locations will have to be investigated using geotechnical drilling.

## 4.4 Model Results

### 4.4.1 Capture Areas for Each Production Well Group

The existing Town of Stephenville well field consists of nine production wells with the current production rate of each well shown in **Table 4.5**. The nine production wells can be separated into four groups based on their locations and production zones/depths (i.e., screen interval of production wells). **Table 4.7** summarizes the four groups of production wells and their maximum recommended production/pumping rates that were used for capture area simulation and **Figures 4.14 to 4.17** shows the steady-state drawdown contour map of capture areas for each of four groups. Besides the pumping rates, the other input parameters for the simulations were the same as those used in the final calibrated model. Note that the actual drawdown at the well is not shown on the drawdown figures.

Well 1 is the only Stephenville well that is completed in the bedrock zone, among the nine production wells, and forms Group 1. The results of the pumping simulation of Well 1 at 1,900 L/min are shown in **Figure 4.14**. The predicted capture area in the bedrock extended beyond Blanche Brook at the south and west bound and spread out in the direction toward Cold Brook at the north-east bound. At the north-west bound, the capture area for Pumping Group 1 alone did not reach the landfill area but was limited by Blanche Brook.

Group 2 consisted of two shallow overburden wells, Well 2 and Well 4, that were constructed at shallow to intermediate depths in the overburden and are located next to a small pond. **Figure 4.15** presents the capture area for the pumping simulation of Well 2 and Well 4 at 800 L/min and 1,200 L/min, respectively. The capture area was limited by Blanche Brook at the south, west and north-west directions, and Cold Brook at the north-east direction. The closely spaced drawdown contours between the pumping wells and the nearby ponds indicate that drawdowns were mitigated by the surface water bodies.

Two wells that are located next to Beaver Pond, Well 5 and Well 6 forms Group 3. Well 5 was installed in the deeper overburden and Well 6 at the intermediate overburden depth. Under the simulation of the two wells pumping at 700 L/min and 310 L/min, respectively (**Figure 4.16**), the drawdowns were mainly limited by Blanche Brook at the south and west direction and spread toward but did not reach Cold Brook at the north-east direction. The closely spaced drawdown contours between the wells and Beaver Pond indicate that the pond was a major water source for Group 3. While this induced infiltration from Beaver Pond reduced the water level in the pond, it did not completely stop the outflow from Beaver Pond.

Group 4 consists of four production wells located along Blanche Brook, Well 7, Well 8, Well 9 and Well 10, which were installed at the intermediate to deeper overburden depths. The results of pumping simulation of Group 4 are shown in **Figure 4.17**. With the maximum recommended pumping rates, 1,400 L/min, 1,500 L/min, 650 L/min and 570 L/min, respectively, the capture areas extended beyond Beaver Pond in the south direction, but did not reach Blanche Brook. In the north-east direction, the drawdown was limited by Cold Brook. However, in the north-west direction, the drawdown extended beyond Blanche Brook and the edge of the capture area reached the landfill.

The capture areas and the drawdown contour map for all nine production wells producing at the recommended maximum pumping rates are shown in **Figure 4.18**. In this simulation, the three production wells at the Kippens well field were also pumping at their recommended maximum rates (Fracflow, 1999a) to simulate their impact on the Stephenville well field. Under the pumping conditions from both well fields, the combined capture areas were controlled and affected by the surface water bodies, such as brooks, streams and ponds. Comparing the local 1 m drawdown contour from the simulation scenario of each group, the 1 m drawdown contour for the current simulation covered the majority of the area between Blanche Brook and Cold Brook. At the north-west side of the well field, the drawdowns extended beyond Blanche Brook and to a 0.05 m contour at the landfill area. In addition, the capture areas from pumping the three Kippens production wells reached to the west side of Blanche Brook and to the landfill area. This simulation indicates that the impact from the landfill area needs to be monitored by installing additional monitoring wells between Blanche Brook and the landfill especially if any combination of pumping rates from the two well fields and new wells result in increasing drawdowns.

### 4.4.2 Computed Travel Times for Flow to Each Production Well

The computed pathways and travel times for each group of wells are presented in **Figures 4.19 to 4.22** using backward tracking method with porosity of 20%. Each pathway shows the travel time symbols ranged from 30 days to 1825 days (5 years).

For Group 1 with the bedrock well (Well 1), the flow paths were not affected by the surface water bodies as shown in **Figure 4.19**. The backward pathways with red triangle symbols indicating five years of travel time were extended in the north direction toward the recharge areas of the drainage basin. Note that one of the flowpath ended at Beaver Pond showing that some of the water was sourced from the pond.

Groups 2 to 4 consist of overburden wells and the simulated flow pathways and travel times are shown in **Figures 4.20 to 4.22**, respectively. Pathways for Group 2 (**Figure 4.20**) shows that the water was recharged both from the upper part of drainage basin and from the nearby pond. However, all flowpaths ended at Beaver Pond for Group 3 as shown in **Figure 4.21** indicating that the main source of water for Group 3 was the pond. The computed pathways for the four wells next to Blanche Brook are presented in **Figure 4.22**. The flowpaths for each well shows that most of the water that moves toward Well 7 and Well 8 is from water that would have discharged to Blanche Brook and that the groundwater produced by Well 9 and Well 10 is derived from the recharge area in the upper part of the drainage basin. Note that some of the pathways terminate in the local recharge area.

### 4.4.3 Impact of Wellfield Production on Baseflow in Blanche Brook

The well yield from the four production wells that are located next to Blanche Brook was, to some extent, contributed by the adjacent brook through induced infiltration as shown in the flow path and particle track simulations. The impact of pumping the four wells on the Blanche Brook baseflow was assessed by calculating the flux through the elements and layers that defined Blanche Brook and the adjacent elements using the 3D flow model. For those flux calculations, the zone of influence under the pumping condition was determined based on the capture area that was defined by the model under the same pumping scenario that was used in the transport modeling. Since the proposed three new well locations are also adjacent to Cold Brook, the same calculation was applied to simulate the impact of each well on the Cold Brook baseflow. **Figure 4.27** shows the two separate flux zones, Zone A on Blanche Brook and Zone B on Cold Brook. The flux for each model slice/layer was calculated by summing all water volumes that moved upward as positive flux and downward as negative flux. **Table 4.8** presents the calculated flux on the top four slices (three layers) for three situations, (1) before pumping condition, i.e., the natural state, (2) when all nine water supply/production wells were pumping, and (3) after adding the three proposed wells at pumping rates of 1,000 L/min (264 USgpm). Before pumping the adjacent wells, both zones showed that the upward flux was much higher than the downward flux which indicated a typical condition of a gaining stream such that the baseflow was flowing into the stream. However, under the pumping condition, the downward flux increased while upward flux decreased. Under this condition, the stream changed from a gaining stream to a losing stream. When the proposed three new wells were add to the pumping scenario, the downward flux increased. The same trend was found in Cold Brook section where the three new wells would be located, with the baseflow being reduced when groundwater withdrawal from the three new wells were added to the existing well field withdrawals.

### 4.4.4 Risks, Wellfield Protection Plan and Exclusion Zones

The three possible locations for new production wells, FW1, FW2 and FW3 are located along the south side of Cold Brook as shown in **Figure 4.10**. Three pumping and drawdown conditions that are created by adding the three new wells to the existing nine production wells are shown in sequence by adding FW2, FW3 and then FW1. **Figure 4.23** shows the capture area when FW2 was added to the existing withdrawals. Then FW3 was added to the system and then finally FW1 was added and all three proposed wells as shown in **Figures 4.25 and 4.26**, respectively. Comparing the groundwater capture area that is produced by pumping the existing production wells to the capture area that is produced by adding the new proposed wells (**Figure 4.18**), it is noted that the capture area after adding FW2 extended beyond Cold Brook in the north direction as well as the 1 m drawdown contour was expanded in the north-east direction toward Cold Brook. By comparison, the drawdown changes in the north-west direction toward the existing landfill area were less significant. As more proposed wells were added, the drawdowns increased in both directions. However, the changes in drawdowns at the north side of the Cold Brook area are more significant than those at the west side of Blanche Brook. This was supported by the particle tracks and pathway simulation results of the three proposed wells (**Figure 4.26**). The flow paths for FW1 and FW3 showed that the water was mainly drawn from the north side of the drainage basin and for FW2 mainly from Cold Brook as well as from the recharge area of the drainage basin.

### 4.4.5 Assessment of Residential Impacts on Water Quality in the Bedrock Aquifer

A transport simulation was completed for a remote residential area at the north side of the main residential area of the Town of Stephenville to assess the impact of residential land use activities, such as septic tank and shallow overburden wells, on the underlying bedrock aquifer. For pathway and particle track simulations, a group of particles were released at each of three locations along Hillier Avenue as shown in **Figure 4.28**. The simulations show that any wastewater that was released at those three locations would migrate down-gradient, partly to local ponds, and would not migrate more than 15 m vertically into the overburden over time (**Figure 4.29**). For most waste water releases, natural attenuation degrades and bioremediates the wastewater as it migrates over distances that can be measured in 10’s of metres. Those releases do not appear to pose any risk to the underlying bedrock aquifer except in those cases where surface casing for wells that have been or will be completed into the bedrock aquifer were not fully sealed from the bottom to the ground surface.

Table 4.1 Details of the layer construction scheme and the hydraulic conductivity values assigned to the each layer with three main areas.

<table>
<tr><th colspan="5"></th><th>Lowland</th><th>Highland</th><th>Rock Exposure / Bedrock</th></tr>
<tr><th>Layer</th><th>Thick-ness (m)</th><th>Depth to Layer Bottom (m)</th><th>Type</th><th>Wells</th><th>Kh / Kv (m/s)</th><th>Kh / Kv (m/s)</th><th>Kh / Kv (m/s)</th></tr>
<tr><td rowspan="2">1</td><td rowspan="2">5</td><td rowspan="2">5</td><td rowspan="2">Upper Till</td><td rowspan="3">--</td><td>5.00E-05 / 2.50E-05</td><td>2.50E-05 / 1.25E-05</td><td rowspan="4">5.00E-06 / 2.50E-06</td></tr>
<tr><td colspan="2">Clay: 1E-8 (bog area)</td></tr>
<tr><td>2</td><td>5</td><td>10</td><td>Lower Till & Clay</td><td>3.00E-05 / 1.50E-05</td><td>2.00E-05 / 1.00E-05</td></tr>
<tr><td>3</td><td>5</td><td>15</td><td>Sand & Clay</td><td>PTW5, PTW6, PTW7</td><td>8.00E-06 / 4.00E-06</td><td>1.00E-05 / 5.00E-06</td></tr>
<tr><td rowspan="2">4</td><td rowspan="2">5</td><td rowspan="2">20</td><td rowspan="4">Basal Sand & Bedrock</td><td rowspan="2">PTW8, PTW11, PTW12</td><td>1.00E-05 / 5.00E-06</td><td>5.00E-06 / 2.50E-06</td><td rowspan="4">2.50E-06 / 1.25E-06</td></tr>
<tr><td colspan="2">Clay: 1E-8 (scattered)</td></tr>
<tr><td>5</td><td>5</td><td>25</td><td>--</td><td>7.50E-06 / 3.75E-06</td><td>2.50E-06 / 1.25E-06</td></tr>
<tr><td>6</td><td>5</td><td>30</td><td>PTW2, PTW9, PTW10</td><td>7.50E-06 / 3.75E-06</td><td>2.50E-06 / 1.25E-06</td></tr>
<tr><td>7</td><td>10</td><td>40</td><td rowspan="2">Sandstone aquifer & Bedrock</td><td rowspan="2">PTW1, KPW1, KPW2, KPW3</td><td rowspan="2">7.00E-05 / 3.50E-05</td><td rowspan="2">7.50E-06 / 3.75E-06</td><td rowspan="2">1.00E-06 / 5.00E-07</td></tr>
<tr><td>8</td><td>10</td><td>50</td></tr>
<tr><td>9</td><td>10</td><td>60</td><td rowspan="8">Basal Bedrock</td><td rowspan="8">--</td><td rowspan="3">5.00E-06 / 2.50E-06</td><td rowspan="3">2.00E-06 / 1.00E-06</td><td rowspan="3">2.50E-07 / 1.25E-07</td></tr>
<tr><td>10</td><td>≥ 20</td><td>≥ 80</td></tr>
<tr><td>11</td><td>≥ 20</td><td>≥ 100</td></tr>
<tr><td>12</td><td>≥ 40</td><td>≥ 140</td><td rowspan="3">1.00E-06 / 5.00E-07</td><td rowspan="3">2.50E-07 / 1.25E-07</td><td rowspan="3">1.00E-07 / 5.00E-08</td></tr>
<tr><td>13</td><td>≥ 40</td><td>≥ 180</td></tr>
<tr><td>14</td><td>≥ 80</td><td>≥ 260</td></tr>
<tr><td>15</td><td>≥ 100</td><td>≥ 360</td><td rowspan="2">1.75E-07 / 8.75E-08</td><td rowspan="2">2.50E-08 / 1.25E-08</td><td rowspan="2">1.25E-08 / 6.25E-09</td></tr>
<tr><td>16</td><td>≥ 140</td><td>500</td></tr>
</table>

Table 4.2 Summary of transmissivity values calculated from a series of aquifer tests conducted in 1999 (Fracflow, 1999b).

| **Model Layer No.** | **Model Layer Thickness (m)** | **Layer Type** | **Well ID** | **Field ID (Drilling)** | **Field [1] Transmissivity T ($m^2/s$)** | **Calculated [2] Model Input Conductivity K (m/s)** |
|---|---|---|---|---|---|---|
| 3 | 5 | Sand | Well 2 | PTW5 | 2.50E-02 | 5.00E-03 |
| | | | Well 4 | PTW6 | 2.20E-02 | 4.40E-03 |
| | | | Well 3 | PTW7 | 4.20E-02 | 8.40E-03 |
| 4 | 5 | Basal Sand | Well 9 | PTW11 | 2.30E-03 | 4.60E-04 |
| | | | Well 10 | PTW12 | 2.80E-03 | 5.60E-04 |
| | | | Well 6 | PTW8 | 4.50E-03 | 9.00E-04 |
| 5 | 10 | Basal Sand | Well 5 | PTW2 | 1.60E-03 | 1.60E-04 |
| | | | Well 7 | PTW9 | 7.80E-03 | 7.80E-04 |
| | | | Well 8 | PTW10 | 7.50E-03 | 7.50E-04 |
| 6 | 10 | Sandstone aquifer | Well 1 | PTW1 | 2.00E-03 | 2.00E-04 |
| 6 & 7 | 10 | Sandstone aquifer | KP-W1 | KP-W1 | 2.30E-03 | 2.30E-04 |
| | | | KP-W2 | KP-W2 | 1.80E-03 | 1.80E-04 |
| | | | KP-W3 | KP-W3 | 9.00E-04 | 9.00E-05 |

[1] Transmissivity values were calculated average values extracted from the previous project based on a series of aquifer tests.

[2] Calculation is based on the measured T and the thickness of the corresponding layer in the model.

Table 4.3 List of the pumping and monitoring wells and corresponding water elevation measured during a series of aquifer tests in each pumping well (Fracflow, 1999b and 2004).

| ***Well ID*** | ***Elev.GS*** ***m*** | ***Water Depth*** ***m*** | ***GW Elevation*** ***m*** |
|---|---|---|---|
| **Production Well** | | | |
| **Well 1** | 70.87 | 5.910 | 64.960 |
| **Well 2** | 65.48 | 0.840 | 64.640 |
| **Well 3** | 66.77 | 1.440 | 65.330 |
| **Well 4** | 67.23 | 1.910 | 65.320 |
| **Well 5** | 73.09 | 8.580 | 64.510 |
| **Well 6** | 73.05 | 7.920 | 65.130 |
| **Well 7** | 80.30 | 0.400 | 79.900 |
| **Well 8** | 79.97 | -0.910 | 80.880 |
| **Well 9** | 84.00 | 0.300 | 83.700 |
| **Well 10** | 86.00 | 0.550 | 85.450 |
| **Monitoring Well** | | | |
| **BB-1** | 70.608 | 1.13 | 69.478 |
| **BB-2** | -- | -- | 66.060 |
| **CB-1** | 98.316 | 2.5 | 95.816 |
| **CB-2** | 99.932 | 2.98 | 96.952 |
| **CB-3** | 85.275 | 0.000 | 85.275 |
| **CB-4** | 98.752 | 0.54 | 98.212 |
| **EXP-1** | 73.274 | 7.7 | 65.574 |
| **EXP-2** | 80.348 | 6.97 | 73.378 |
| **EXP-3** | 70.715 | 4.75 | 65.965 |
| **EXP-4** | 100.175 | 2.5 | 97.675 |
| **LF1-1** | 100.351 | 5.42 | 94.931 |
| **LF1-2** | 99.636 | 4.96 | 94.676 |
| **LF1-3** | 95.506 | 2.32 | 93.186 |
| **LF1-4** | 95.266 | 14.5 | 80.766 |
| **LF2-1** | 68.100 | 12.59 | 55.510 |
| **LF2-2** | 68.654 | 10.91 | 57.744 |
| **LF2-4** | 68.104 | 11.67 | 56.434 |
| **SM-1** | -- | -- | 81.410 |
| **PTW-3** | NA | NA | NA |
| **PTW-4** | NA | NA | NA |
| **Kippens Well** | | | |
| **KP-W1** | 78.64 | 5.6 | 73.040 |
| **KP-W2** | 80.988 | 3.7 | 77.288 |
| **KP-W3** | 75.432 | 0.8 | 74.632 |

Table 4.4 Selected calibration scenarios for the recharge assignments.

| | **Total Precipitation (mm/yr) : 1237.35** | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| ***Calibration No.*** | ***Zone 1*** | | ***Zone 2*** | | ***Zone 3*** | | ***Zone 4*** | |
| | ***%*** | ***mm/yr*** | ***%*** | ***mm/yr*** | ***%*** | ***mm/yr*** | ***%*** | ***mm/yr*** |
| **11** | 23% | 280.0 | 16% | 200.0 | 16% | 200.0 | 23% | 280.0 |
| **12** | 23% | 280.0 | 18% | 220.0 | 10% | 120.0 | 23% | 280.0 |
| **13** | 27% | 330.0 | 22% | 270.0 | 20% | 245.0 | 30% | 370.0 |
| **14 (Final)** | 22% | 270.0 | 15% | 185.0 | 7% | 85.0 | 28% | 345.0 |

Table 4.5 Summary of flowrate data compiled from well records collected between 2016 and 2021 from the nine production wells.

| Town Well ID | Drill Well ID | Average Flowrate for 2016 - 2021 | | Average Flowrate[1] for Month of Jan. 2021 | | Flowrate on Jan. 28, 2021 | | Flowrate[2] For Model Calibration | |
|---|---|---|---|---|---|---|---|---|---|
| | | L/min | GPM | L/min | GPM | L/min | GPM | L/min | GPM |
| Well 1 | PTW-1 | 1076 | 284 | 1211 | 320 | 1302 | 344 | 1079 | 285 |
| Well 2 | PTW-5 | 136 | 36 | 163 | 43 | 180 | 48 | 132 | 35 |
| Well 3 | PTW-7 | na | na | na | na | na | na | na | na |
| Well 4 | PTW-6 | 1069 | 282 | 1287 | 340 | 1396 | 369 | 1098 | 290 |
| Well 5 | PTW-2 | 288 | 76 | 227 | 60 | 242 | 64 | 303 | 80 |
| Well 6 | PTW-8 | 62 | 16 | 76 | 20 | 83 | 22 | 62 | 17 |
| Well 7 | PTW-9 | 868 | 229 | 946 | 250 | 1030 | 272 | 867 | 229 |
| Well 8 | PTW-10 | 687 | 181 | 757 | 200 | 813 | 215 | 687 | 182 |
| Well 9 | PTW-11 | 329 | 87 | 284 | 75 | 307 | 81 | 322 | 85 |
| Well 10 | PTW-12 | 310 | 82 | 246 | 65 | 241 | 64 | 310 | 82 |

[1] Flowrates for Jan. 2021 are determined using both the average of Jan 2021 and the day flowrate on Jan. 28, 2021.

[2] Simulation flowrates are determined considering 'median' and 'average' values of the 5 year flow records.

Table 4.6 List of monitoring wells with depths to water measured in 1999 and January 2021.

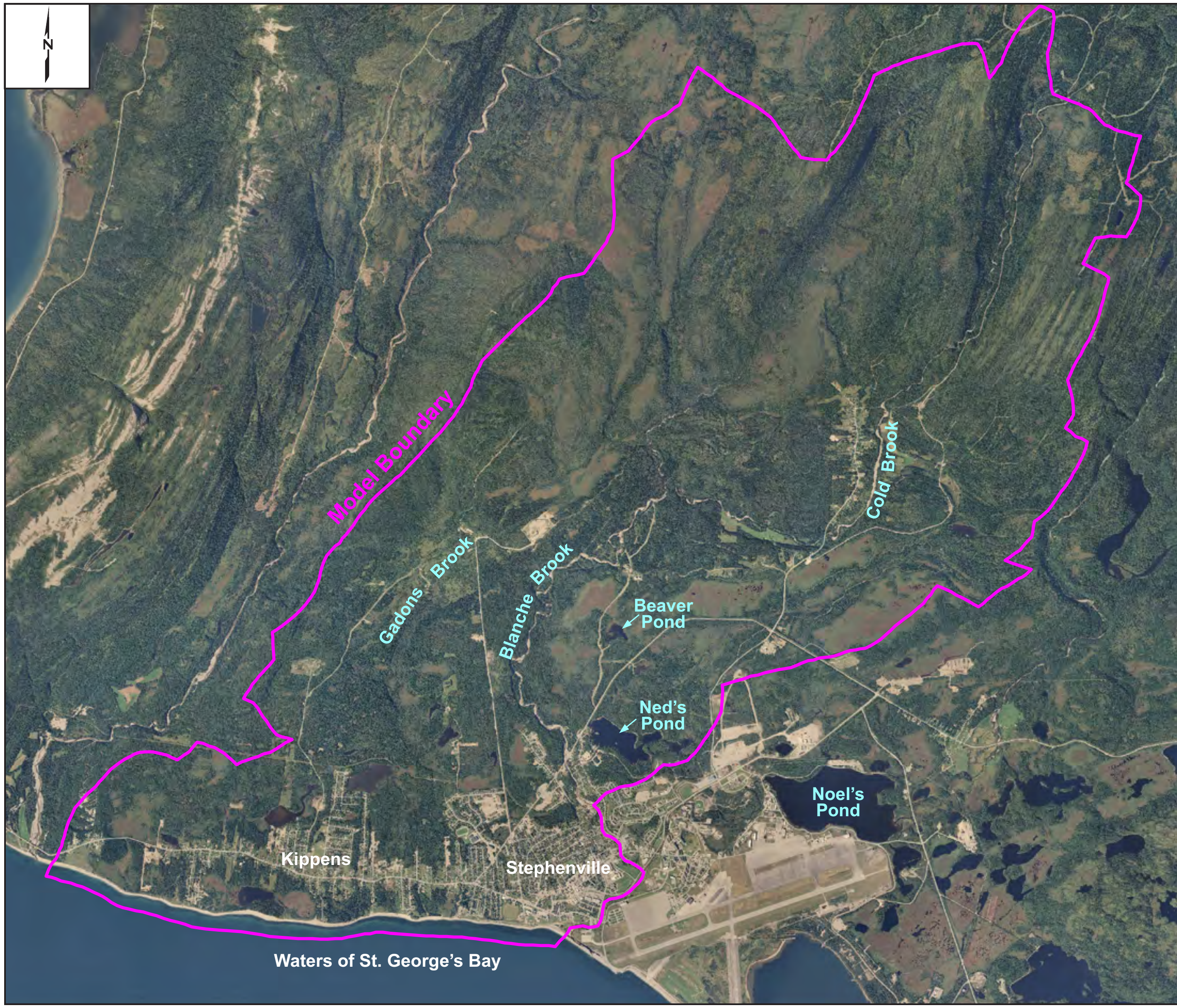
| Well ID | Depth to Water from GS (m) | | Difference in Depth to Water (m) |
|---|---|---|---|
| | 1999 | 2021 | |
| EXP-1 | 8.180 | 10.022 | -1.842 |
| EXP-3 | 5.290 | 7.151 | -1.861 |
| LF1-3 | 2.320 | 2.303 | 0.017 |
| LF1-4 | 15.380 | 6.412 | 8.968 |
| LF2-4 | 11.670 | 11.643 | 0.027 |
| Well 3 (PTW-7) | 1.440 | 2.882 | -1.442 |
| PTW-3 | na | 9.906 | na |
| PTW-4 | 4.780 | 15.113 | -10.333 |
| SM-1 | 2.350 | 4.898 | -2.548 |

Table 4.7 Group of production wells for capture area and travel time simulations.

| Town Well ID | Drill Well I.D. | Group | Recommended Flowrate for Model Simulation | | |
|---|---|---|---|---|---|
| | | | L/min | GPM | $m^3/d$ |
| Well 1 | PTW-1 | Group 1 | 1,900 | 501.9 | 2,736.0 |
| Well 2 | PTW-5 | Group 2 | 800 | 211.3 | 1,152.0 |
| Well 4 | PTW-6 | | 1,200 | 317.0 | 1,728.0 |
| Well 5 | PTW-2 | Group 3 | 700 | 184.9 | 1,008.0 |
| Well 6 | PTW-8 | | 310 | 81.9 | 446.4 |
| Well 7 | PTW-9 | Group 4 | 1,400 | 369.8 | 2,016.0 |
| Well 8 | PTW-10 | | 1,500 | 396.3 | 2,160.0 |
| Well 9 | PTW-11 | | 650 | 171.7 | 936.0 |
| Well 10 | PTW-12 | | 570 | 150.6 | 820.8 |
| FW1 | | Proposed Well | 1,000 | 264.2 | 1,440.0 |
| FW2 | | | 1,000 | 264.2 | 1,440.0 |
| FW3 | | | 1,000 | 264.2 | 1,440.0 |
| KP-W1 | | Kippens Production Well | 350 | 92.5 | 504.0 |
| KP-W2 | | | 750 | 198.1 | 1,080.0 |
| KP-W3 | | | 550 | 145.3 | 792.0 |

Table 4.8 Flux calculation results at the top four slices of the 3D model from two zones along Blanche Brook and Cold Brook.

| Layer No | Slice No | Slice Depth m | **Before Pumping** | | | | **Pumping Production Wells** | | | | **Adding Proposed Wells** | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Up | Down | **Total** | | Up | Down | **Total** | | Up | Down | **Total** | |
| | | | $m^3/d$ | | **$m^3/d$** | **$m^3/s$** | $m^3/d$ | | **$m^3/d$** | **$m^3/s$** | $m^3/d$ | | **$m^3/d$** | **$m^3/s$** |
| **Zone A: Blanche Brook - Well 7, Well 8, Well 9 and Well 10** | | | | | | | | | | | | | | |
| | S1 | 0 | 2949 | -506 | **2443** | **0.0283** | 1682 | -1689 | **-7** | **-8E-05** | 1521 | -1770 | **-249** | **-0.003** |
| 1 | | | | | | | | | | | | | | |
| | S2 | 5 | 2226 | -239 | **1987** | **0.023** | 1156 | -1172 | **-16** | **-2E-04** | 1024 | -1237 | **-213** | **-0.002** |
| 2 | | | | | | | | | | | | | | |
| | S3 | 10 | 1427 | -52 | **1375** | **0.0159** | 677 | -647 | **30** | **0.0003** | 582 | -684 | **-102** | **-0.001** |
| 3 | | | | | | | | | | | | | | |
| | S4 | 15 | 1018 | -7 | **1012** | **0.0117** | 475 | -451 | **24** | **0.0003** | 401 | -473 | **-72** | **-8E-04** |
| **Zone B: Cold Brook - FW1, FW2 and FW3** | | | | | | | | | | | | | | |
| | S1 | 0 | 1307 | -176 | **1131** | **0.0131** | 1233 | -196 | **1037** | **0.012** | 364 | -1027 | **-664** | **-0.008** |
| 1 | | | | | | | | | | | | | | |
| | S2 | 5 | 907 | -107 | **800** | **0.0093** | 852 | -125 | **727** | **0.0084** | 216 | -774 | **-558** | **-0.006** |
| 2 | | | | | | | | | | | | | | |
| | S3 | 10 | 479 | -49 | **430** | **0.005** | 446 | -62 | **384** | **0.0044** | 96 | -474 | **-378** | **-0.004** |
| 3 | | | | | | | | | | | | | | |
| | S4 | 15 | 242 | -24 | **217** | **0.0025** | 223 | -35 | **188** | **0.0022** | 44 | -303 | **-259** | **-0.003** |




Note: Base orthorectified images/maps by Department of Fisheries and Land Resources (2937_Z3, 2938_Z3, and 3038_Z3).

Figure 4.1

General location map of the project site with the model/drainage boundaries, Stephenville, NL.

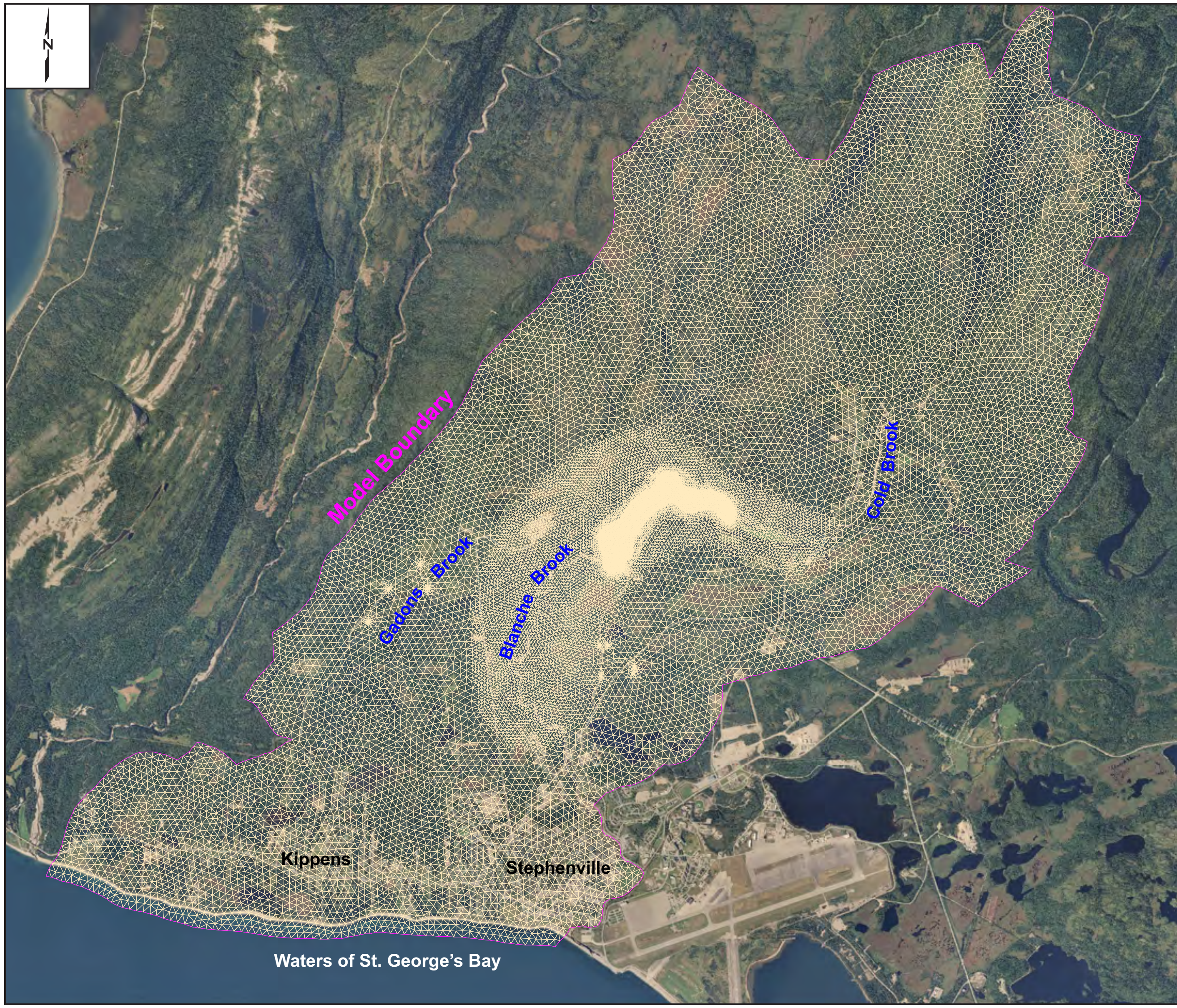
Project No. 555

Location Stephenville, NL

Document Reference FFC-NL-555-004

Date June 2021

FFC




Note: Base orthorectified images/maps by Department of Fisheries and Land Resources (2937_Z3, 2938_Z3, and 3038_Z3).

Figure 4.2

Model boundary with generated mesh for 3D flow and transport model simulation at the project site in Stephenville, NL.

| | |
|---|---|
| Project No. | 555 |
| Location | Stephenville, NL |
| Document Reference | FFC-NL-555-004 |
| Date | June 2021 |

FFC

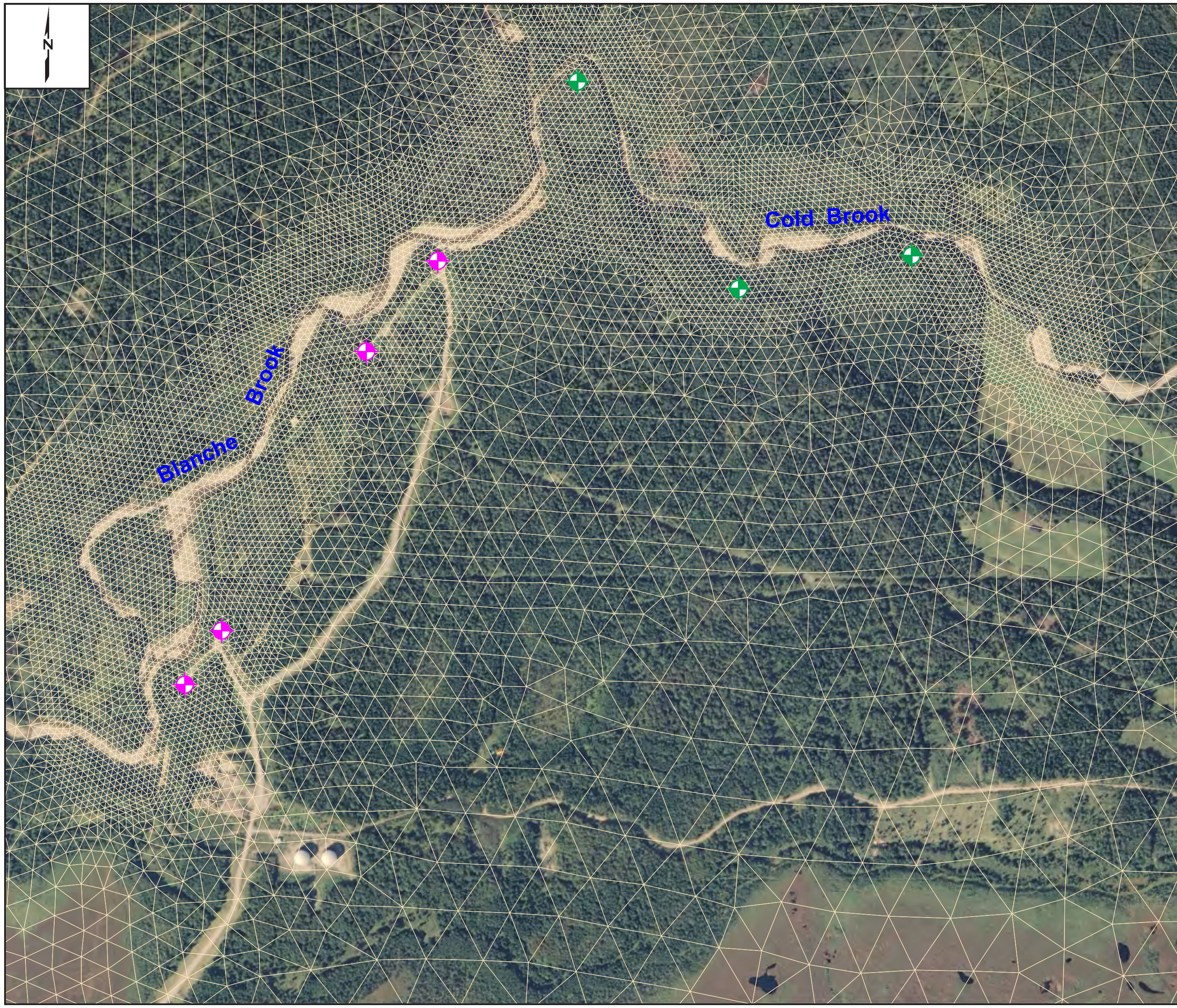



Figure 4.3

FEFLOW model mesh scheme showing refined meshes along the Blanche Brook and coarse meshes with distances from the brook.

| | |
|---|---|
| Project No. | 555 |
| Location | Stephenville, NL |
| Document Reference | FFC-NL-555-004 |
| Date | June 2021 |

FFC


Legend

3D model mesh

0 10 20 30

Scale, m

Note: Base orthorectified images/maps by Department of Fisheries and Land Resources (2937_Z3, 2938_Z3, and 3038_Z3).

Figure 4.4

FEFLOW mesh (plan view) showing an example of refined meshes around a production well, Well 9.

Project No.
555

Location
Stephenville, NL

Document Reference
FFC-NL-555-004

Date
June 2021

FFC

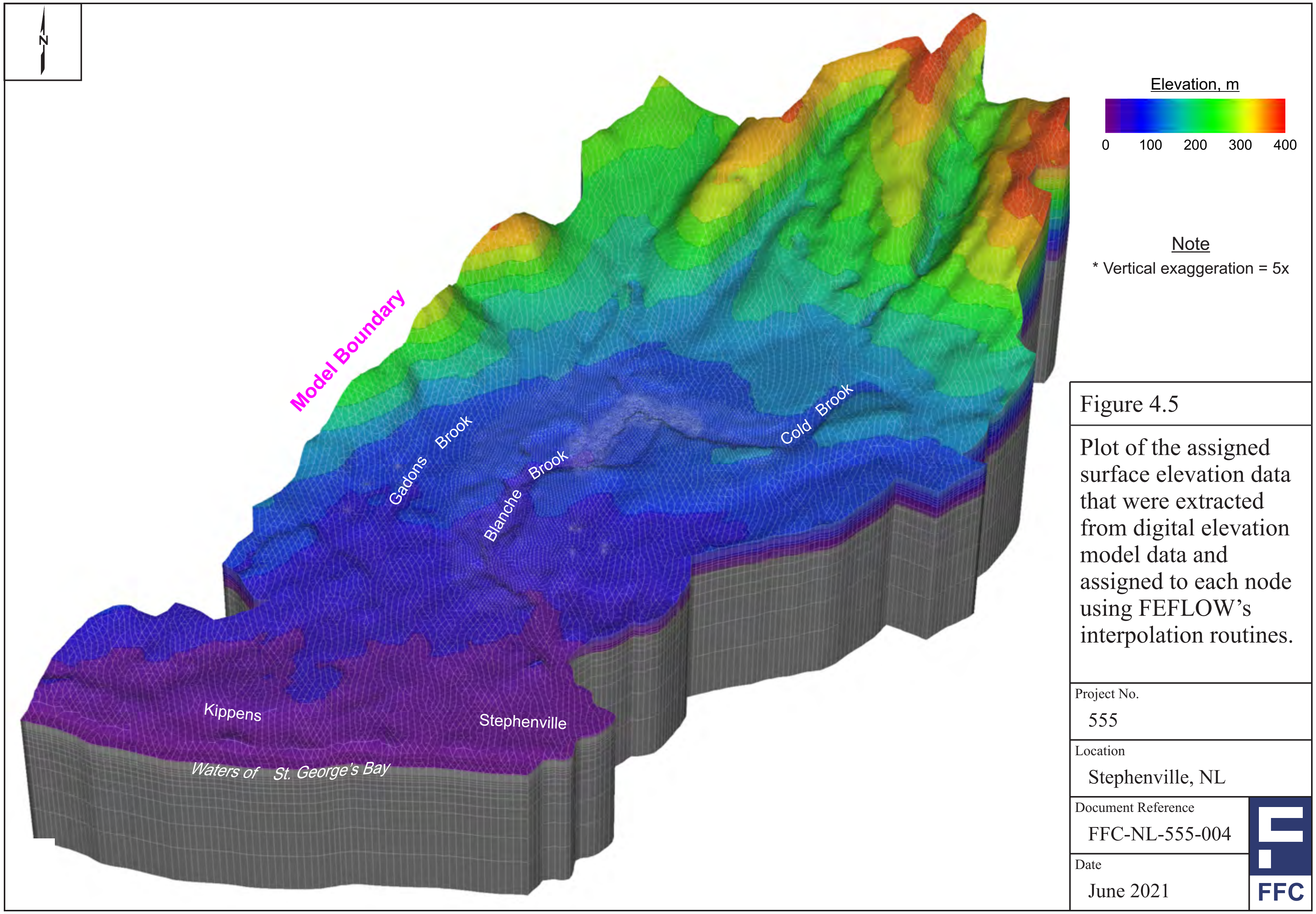
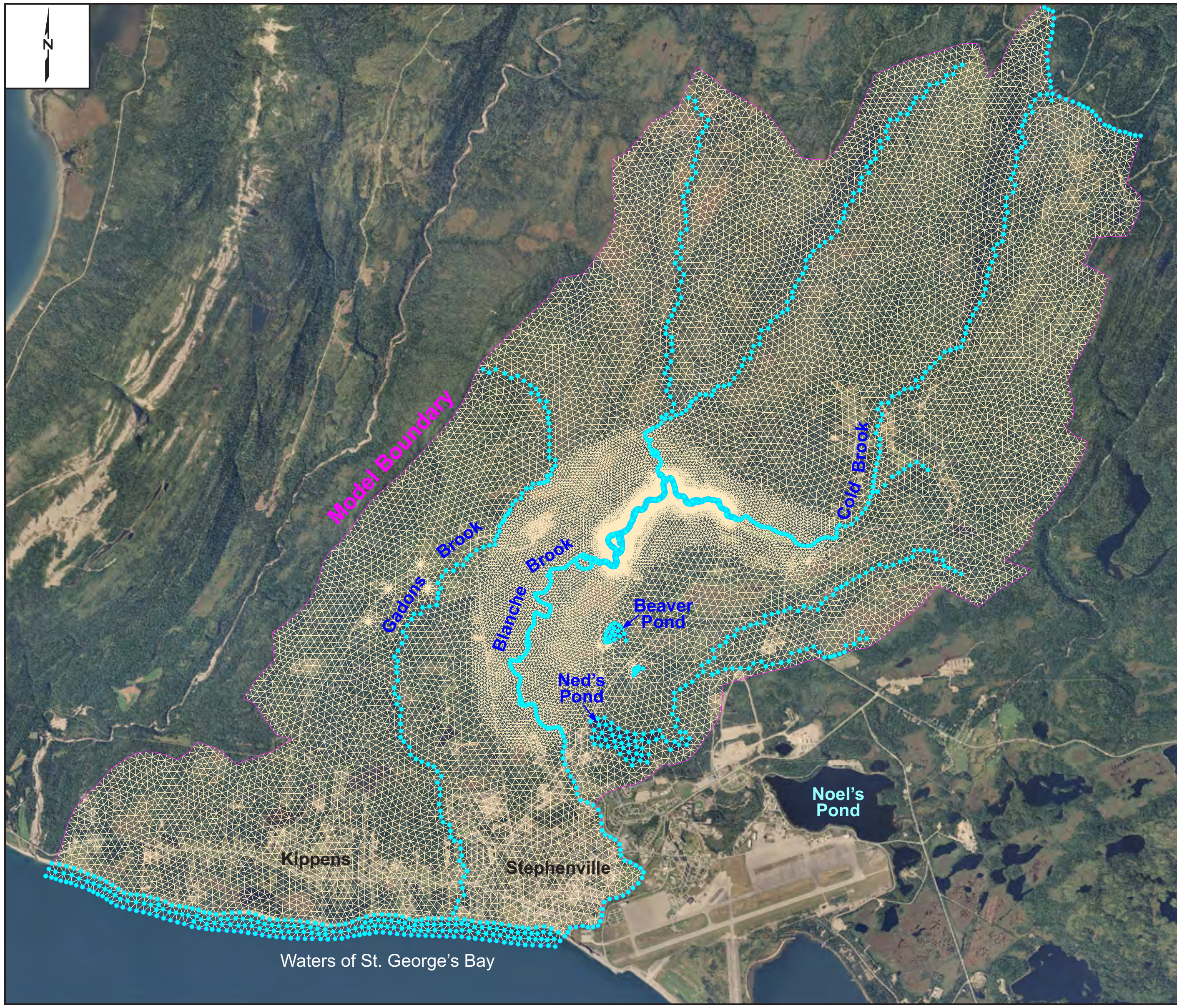



Figure 4.5

Plot of the assigned surface elevation data that were extracted from digital elevation model data and assigned to each node using FEFLOW's interpolation routines.

| | |
|---|---|
| Project No. | 555 |
| Location | Stephenville, NL |
| Document Reference | FFC-NL-555-004 |
| Date | June 2021 |

FFC




<u>Legend</u>

Location of boundary conditions

3D model mesh

0 1 2 3

<u>Scale, km</u>

<u>Note</u>: Base orthorectified images/maps by Department of Fisheries and Land Resources (2937_Z3, 2938_Z3, and 3038_Z3).

Figure 4.6

Selected surface waterbodies, such as brooks/streams and ponds, that were used as constant head boundary conditions in the model.

| | |
|---|---|
| Project No. | 555 |
| Location | Stephenville, NL |
| Document Reference | FFC-NL-555-004 |
| Date | June 2021 |

FFC

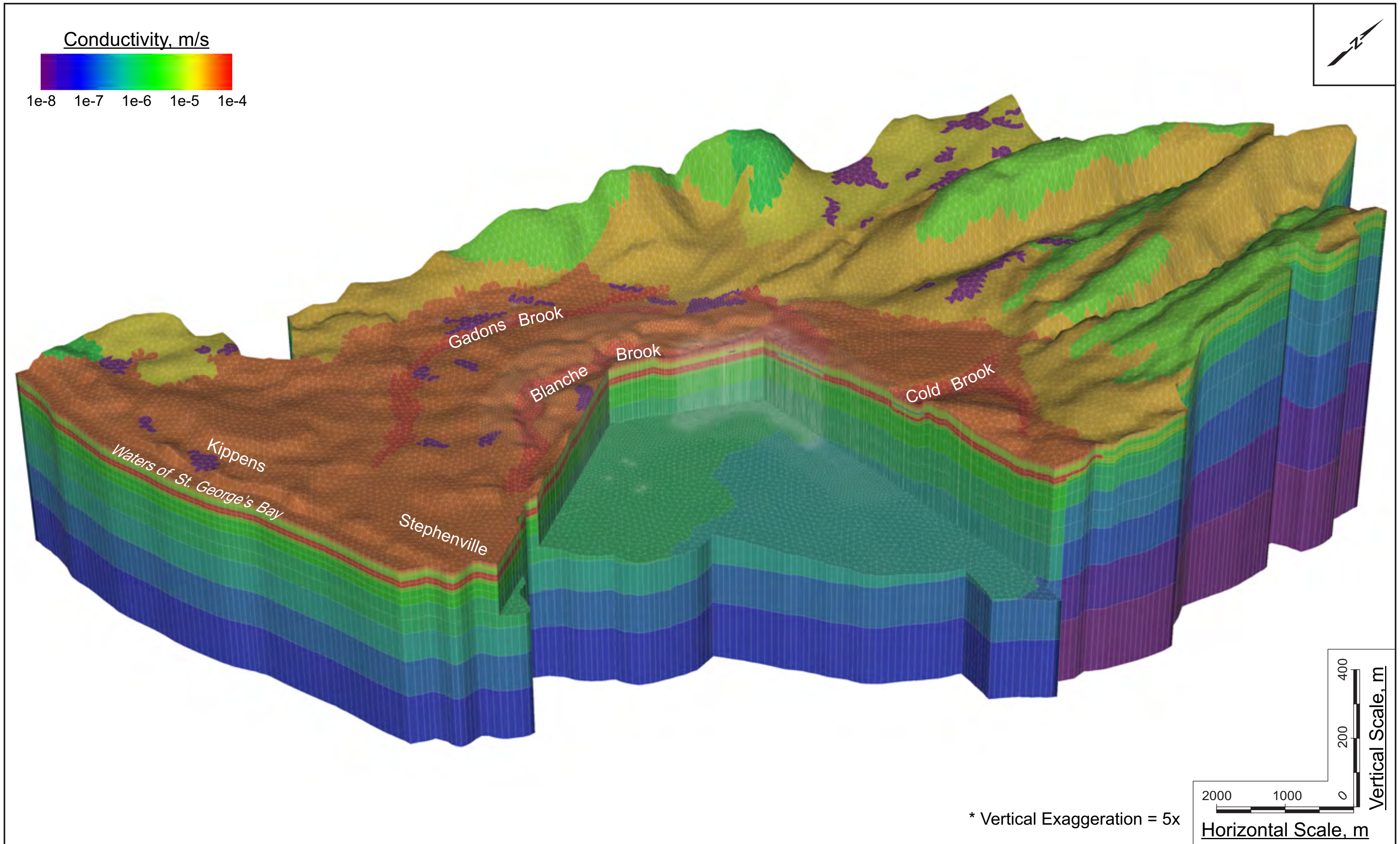
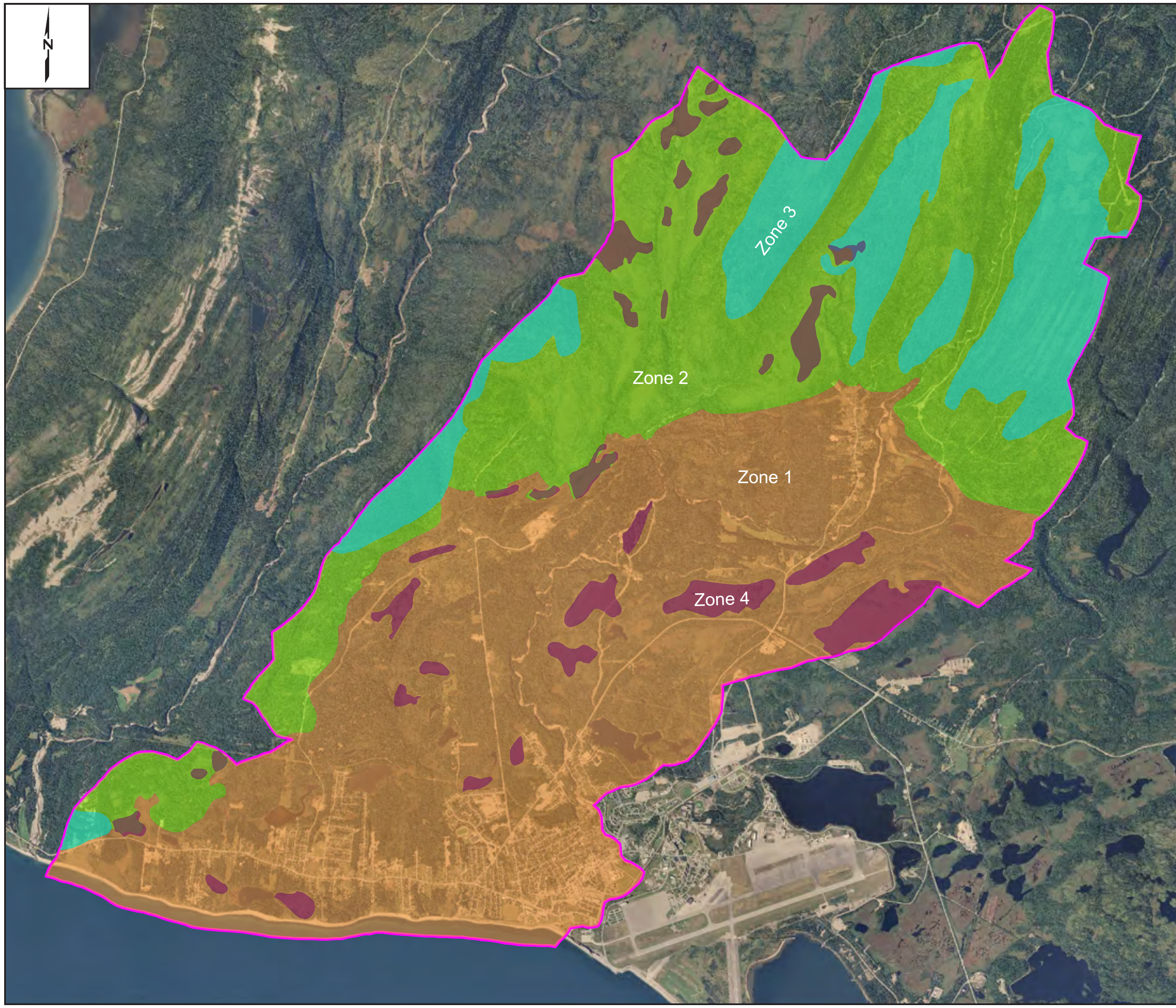



Figure 4.7 Cross-section of the 3D numerical model showing the assigned hydraulic conductivity values for the layers.

| Project No. | Document Reference |
|---|---|
| 555 | FFC-NL-555-004 |
| **Location** | **Date** |
| Stephenville, NL | June 2021 |

FFC




Legend

Zone 1
Zone 2
Zone 3
Zone 4

0 1 2 3
Scale, km

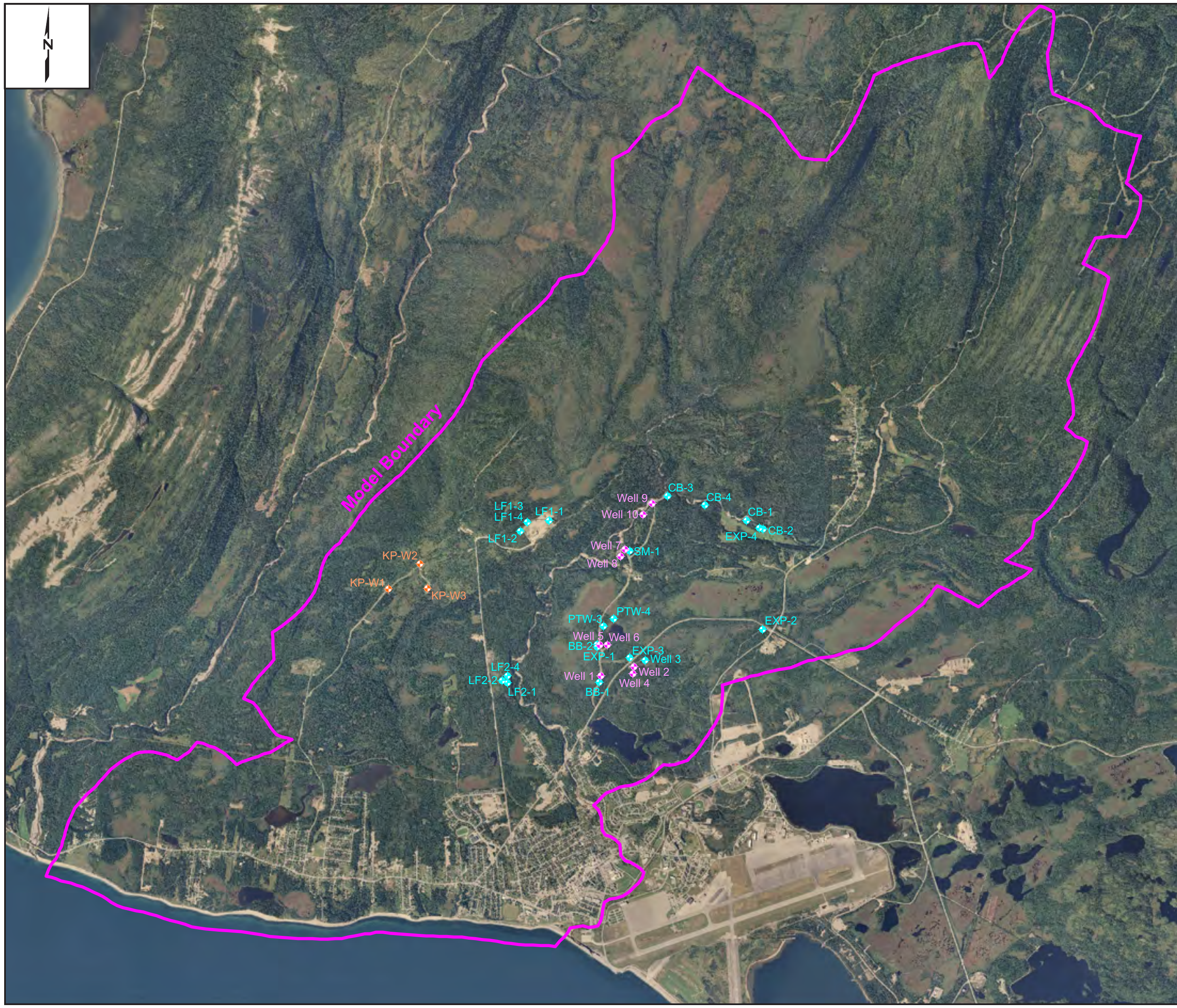
Note: Base orthorectified images/maps by Department of Fisheries and Land Resources (2937_Z3, 2938_Z3, and 3038_Z3).

Figure 4.8

Four sub-divided zones for recharge assignments.

| | |
|---|---|
| Project No. | 555 |
| Location | Stephenville, NL |
| Document Reference | FFC-NL-555-004 |
| Date | June 2021 |

FFC




Legend

Well 1 Production well - Stephenville

BB-1 Monitoring well

KP-W2 Production well - Kippens

0 1 2 3

Scale, km

Note: Base orthorectified images/maps by Department of Fisheries and Land Resources (2937_Z3, 2938_Z3, and 3038_Z3).

Figure 4.9

Production and monitoring well locations within the Stephenville model domain including three Kippens production wells.

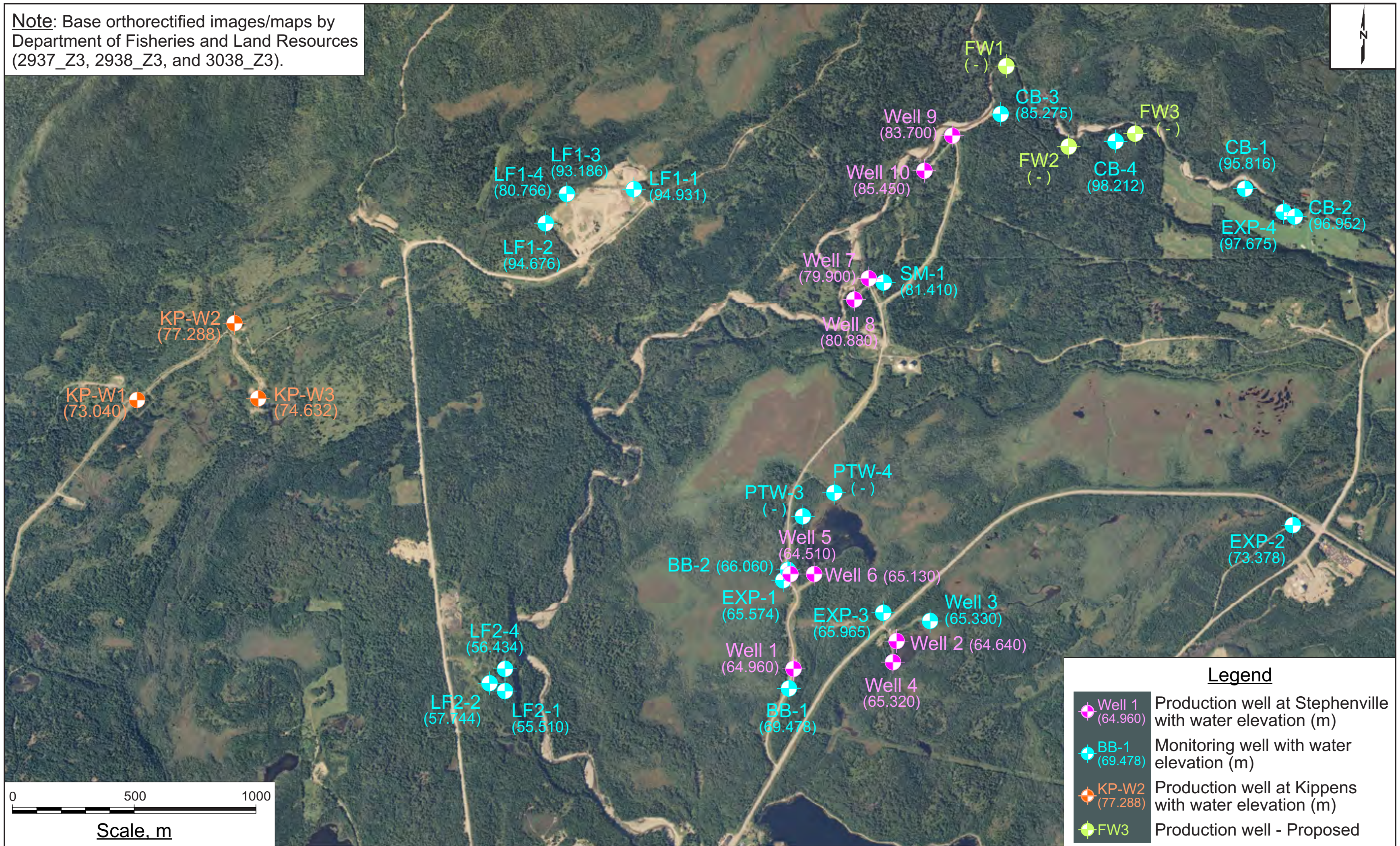
Project No.
555

Location
Stephenville, NL

Document Reference
FFC-NL-555-004

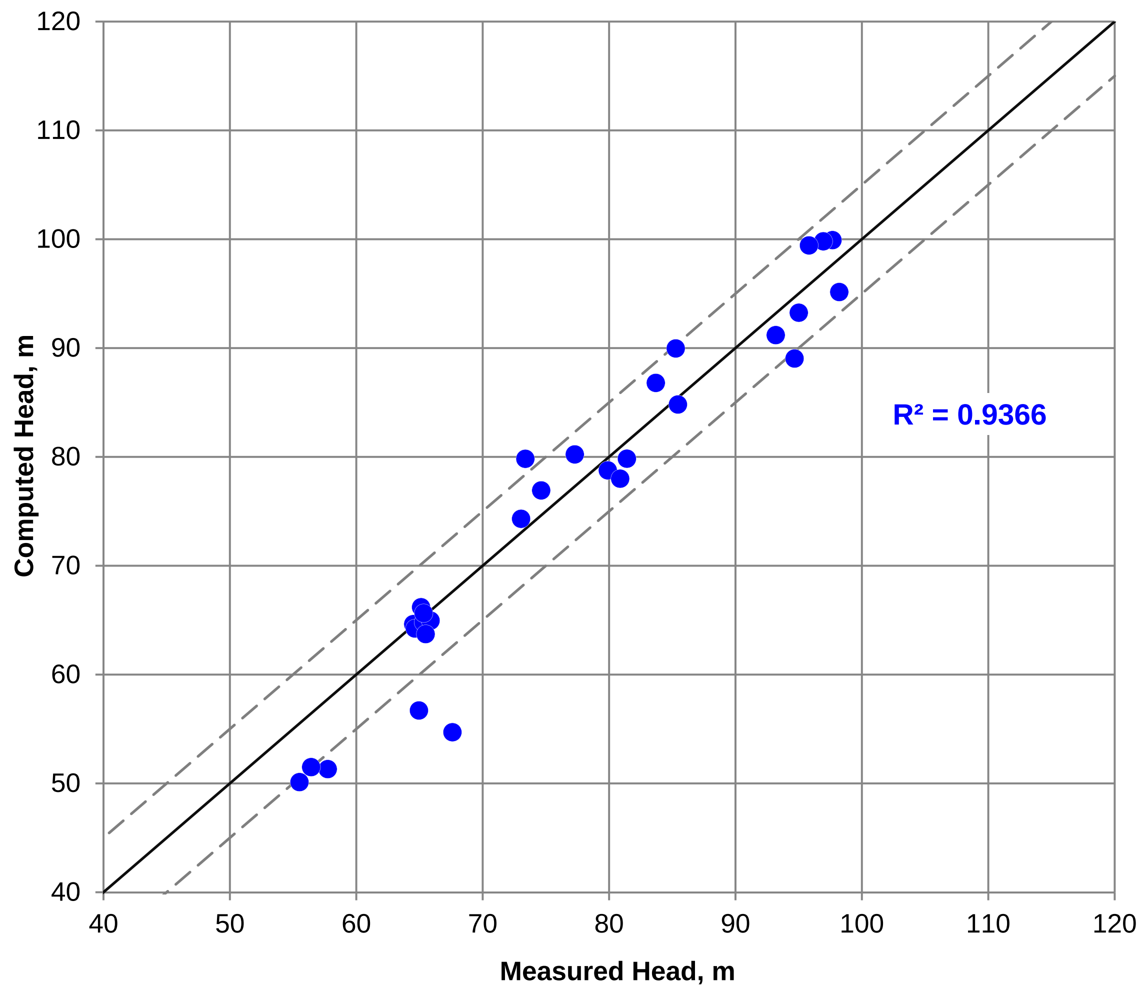
Date
June 2021

FFC



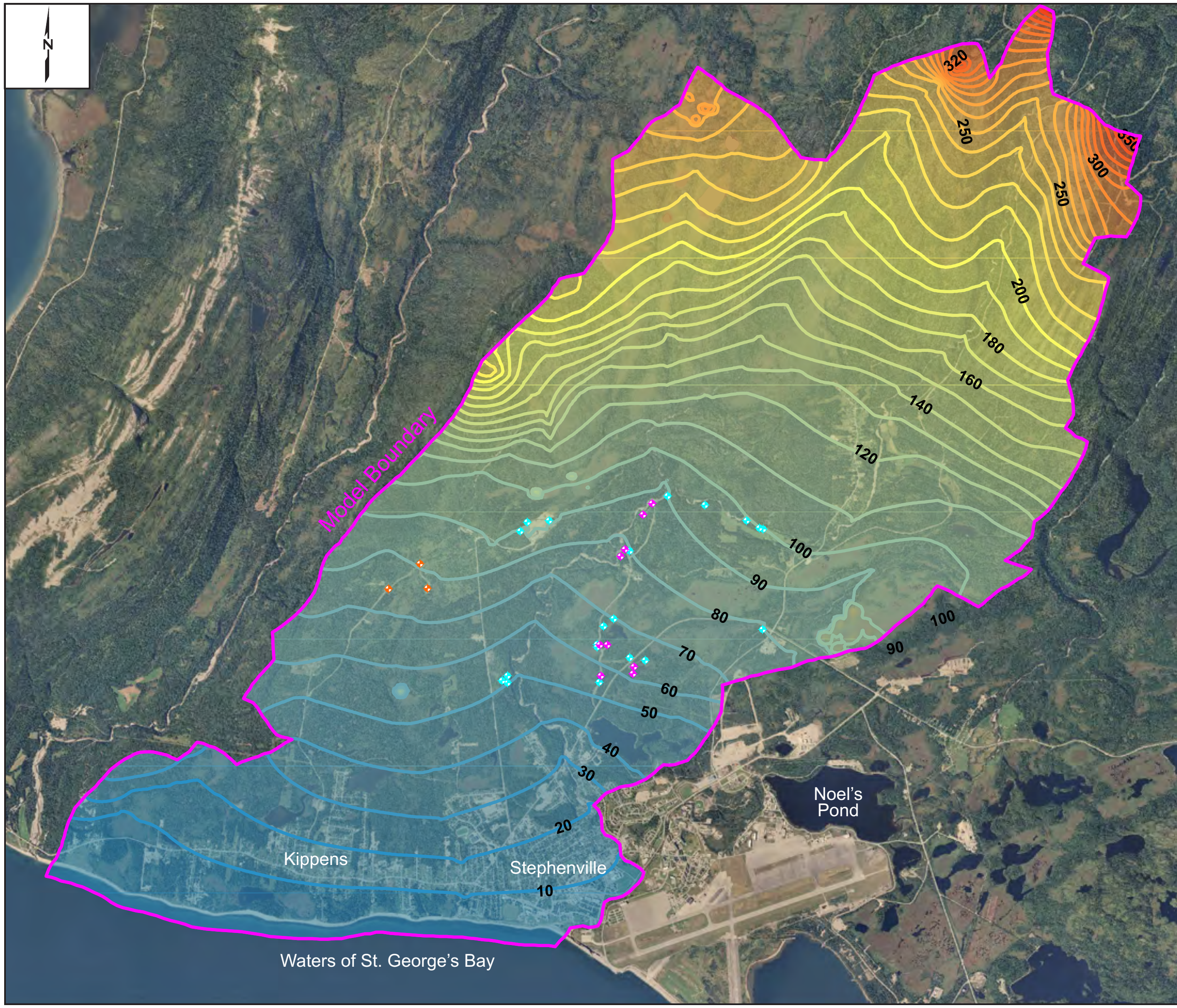

| | | |
|---|---|---|
| Figure 4.10 Production and monitoring well locations with compiled hydraulic head data from existing reports. | Project No. 555 | Document Reference FFC-NL-555-004 |
| | Location Stephenville, NL | Date June 2021 |

FFC




| Figure 4.11 Plot of measured versus the hydraulic heads that were computed by the 3D model at the available locations under a non-pumping condition. | Project No. 555 | Document Reference FFC-NL-555-004 |
|---|---|---|
| | Location Stephenville, NL | Date June 2021 |

FFC






Note: Base orthorectified images/maps by Department of Fisheries and Land Resources (2937_Z3, 2938_Z3, and 3038_Z3).

Figure 4.12

Contour map of the computed water elevation (hydraulic head) in metres under a steady-state non-pumping condition.

| | |
|---|---|
| Project No. | 555 |
| Location | Stephenville, NL |
| Document Reference | FFC-NL-555-004 |
| Date | June 2021 |

FFC

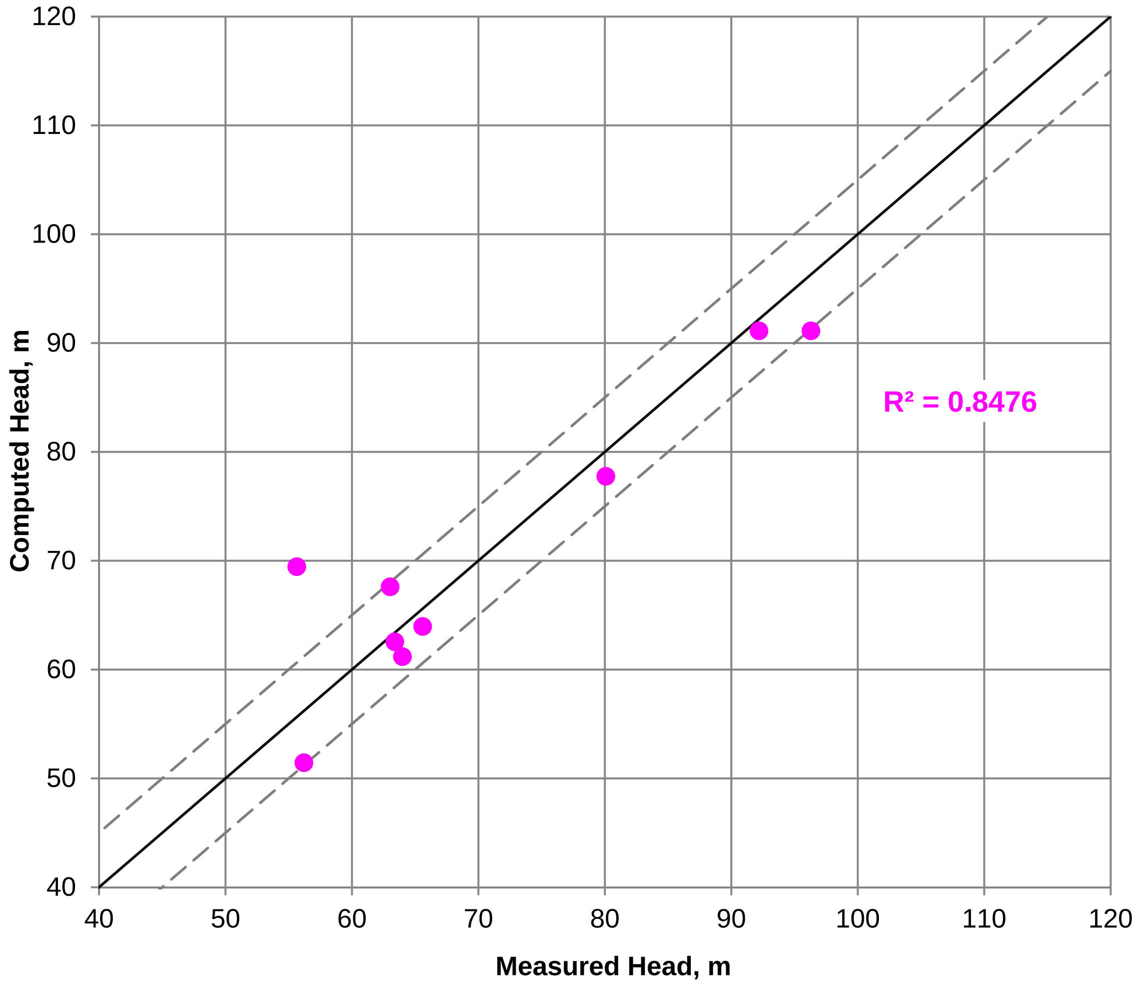
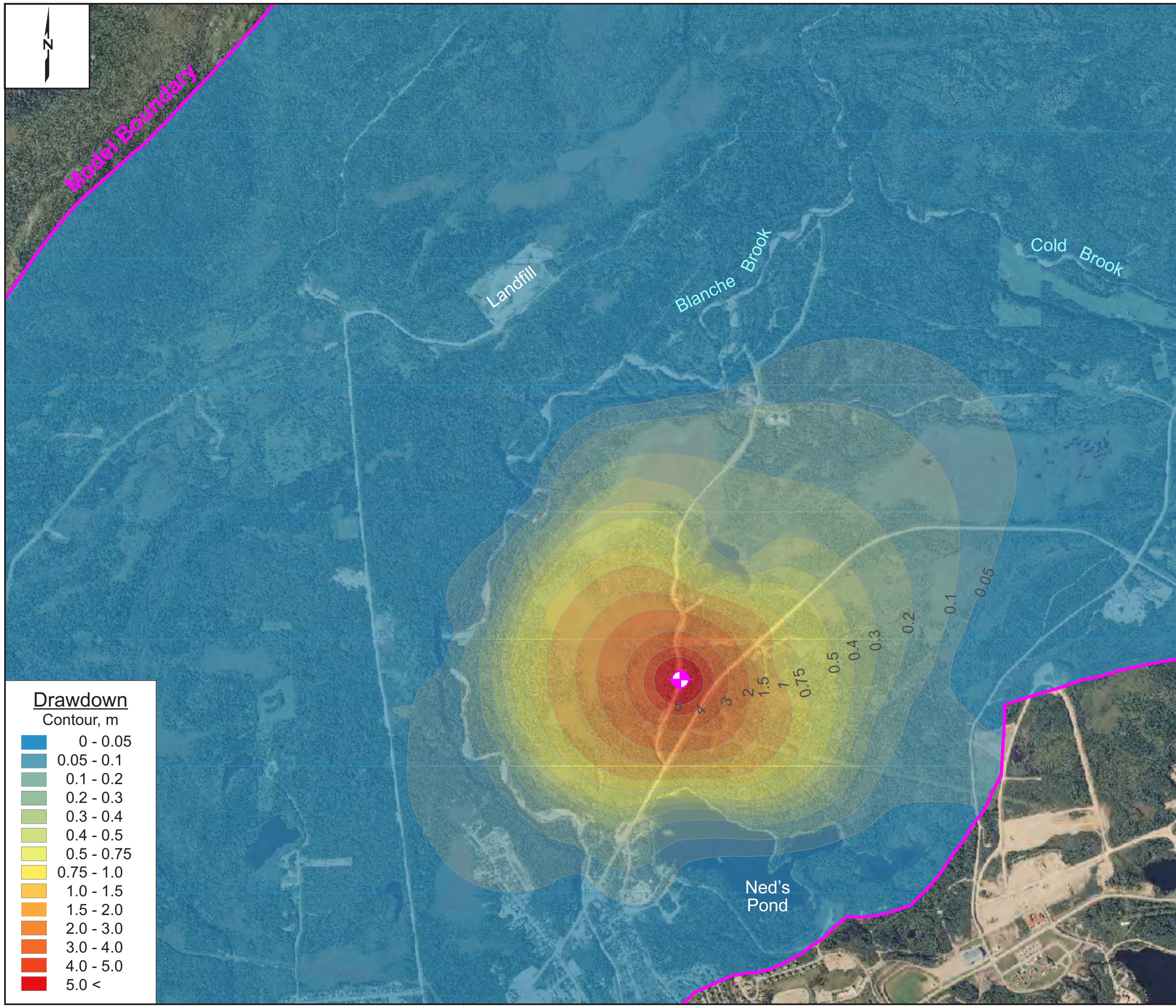



Figure 4.13 Plot of measured versus the hydraulic heads that were computed by the 3D model at the available locations under a pumping condition.

| | |
|---|---|
| Project No. 555 | Document Reference FFC-NL-555-004 |
| Location Stephenville, NL | Date June 2021 |

FFC




**Legend**

Production well under pumping condition for simulation

0 500 1000

Scale, m

Note: Base orthorectified images/maps by Department of Fisheries and Land Resources (2937_Z3, 2938_Z3, and 3038_Z3).

Figure 4.14

Capture area predicted by the model with steady-state drawdown contours in metres for a scenario of pumping Well 1 at 1,900 L/min.

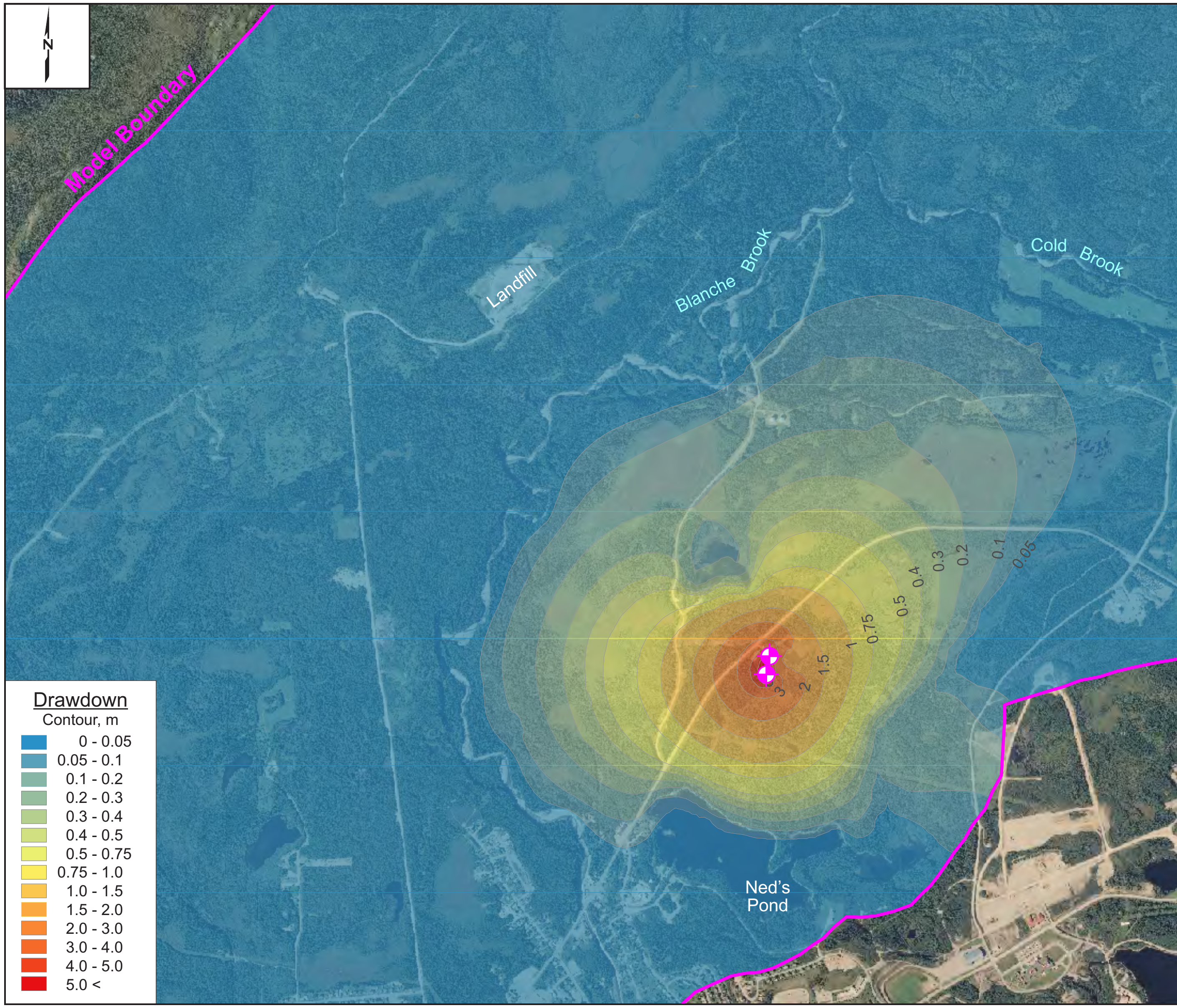
Project No.
555

Location
Stephenville, NL

Document Reference
FFC-NL-555-004

Date
June 2021

FFC




Legend

Production well under pumping condition for simulation

0 500 1000

Scale, m

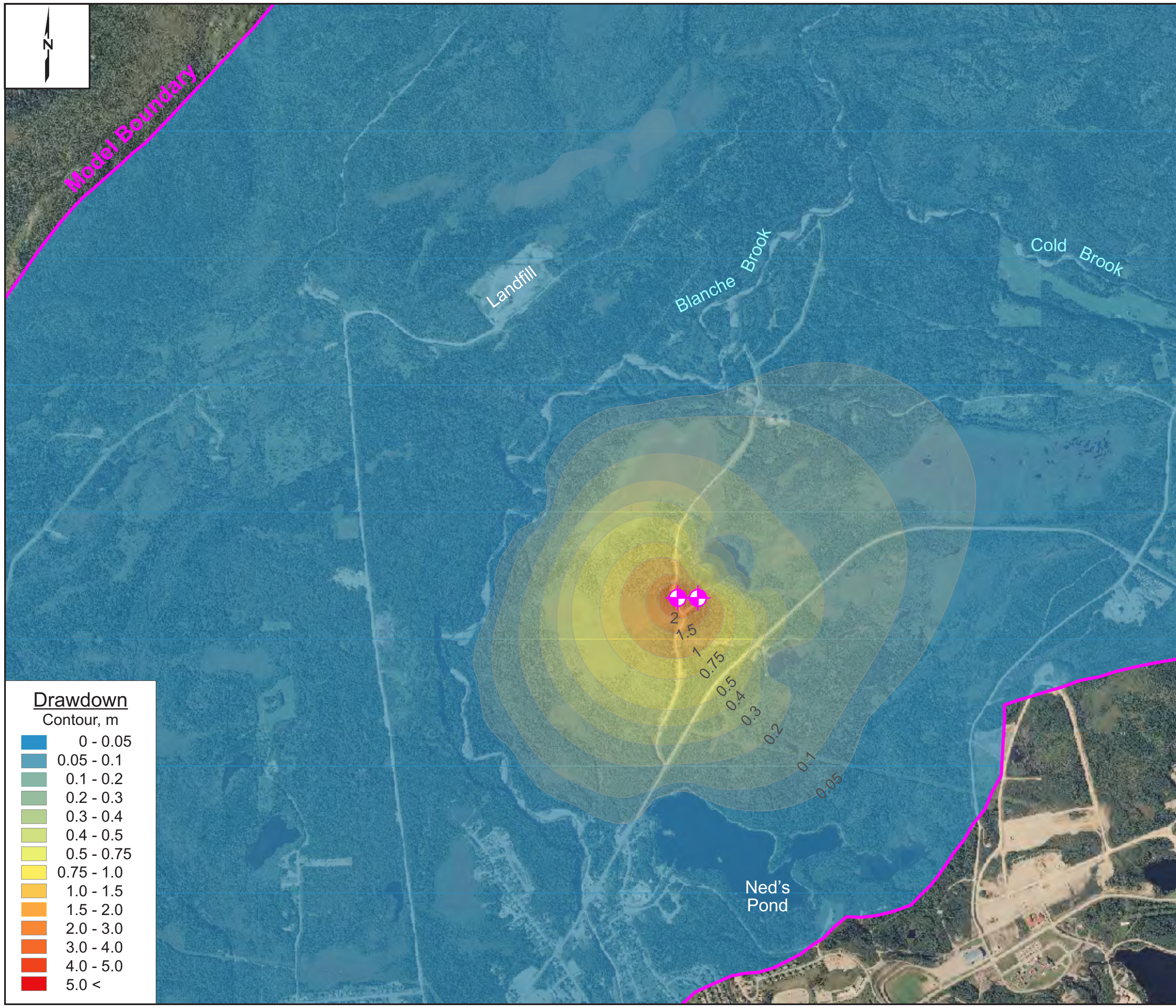
Note: Base orthorectified images/maps by Department of Fisheries and Land Resources (2937_Z3, 2938_Z3, and 3038_Z3).

Figure 4.15

Capture area predicted by the model with steady-state drawdown contours in metres for a scenario of pumping Well 2 and Well 4 at 800 L/min and 1,200 L/min, respectively.

| | |
|---|---|
| Project No. | 555 |
| Location | Stephenville, NL |
| Document Reference | FFC-NL-555-004 |
| Date | June 2021 |

FFC




Legend

Production well under pumping condition for simulation

0 500 1000

Scale, m

Note: Base orthorectified images/maps by Department of Fisheries and Land Resources (2937_Z3, 2938_Z3, and 3038_Z3).

Figure 4.16

Capture area predicted by the model with steady-state drawdown contours in metres for a scenario of pumping Well 5 and Well 6 at 700 L/min and 310 L/min, respectively.

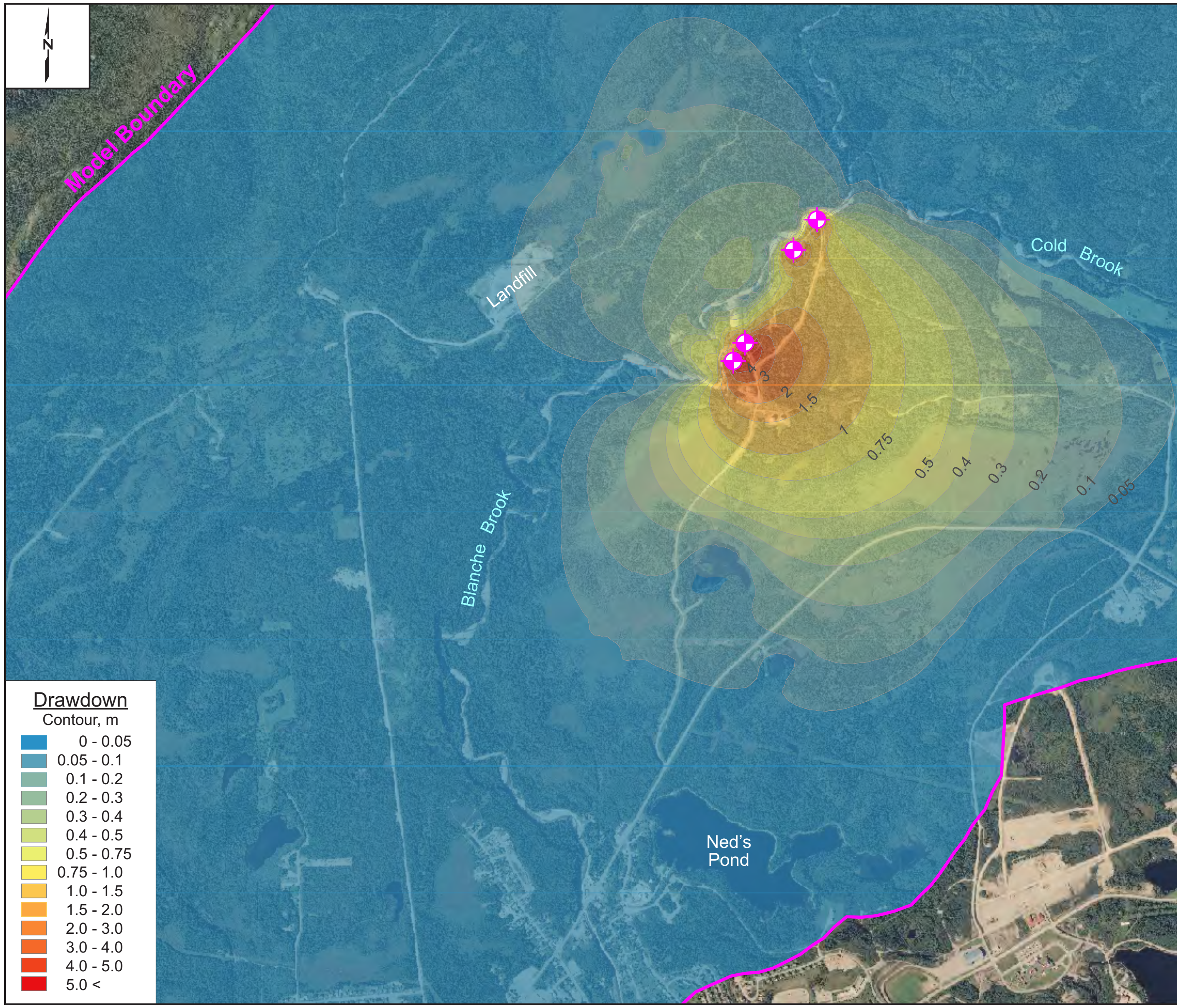
Project No.
555

Location
Stephenville, NL

Document Reference
FFC-NL-555-004

Date
June 2021

FFC




**Legend**

Production well under pumping condition for simulation

0 500 1000

Scale, m

Note: Base orthorectified images/maps by Department of Fisheries and Land Resources (2937_Z3, 2938_Z3, and 3038_Z3).

Figure 4.17

Capture area predicted by the model with steady-state drawdown contours in metres for a scenario of pumping Well 7, Well 8, Well 9 and Well 10 at 1,400, 1,500, 650 and 570 L/min, respectively.

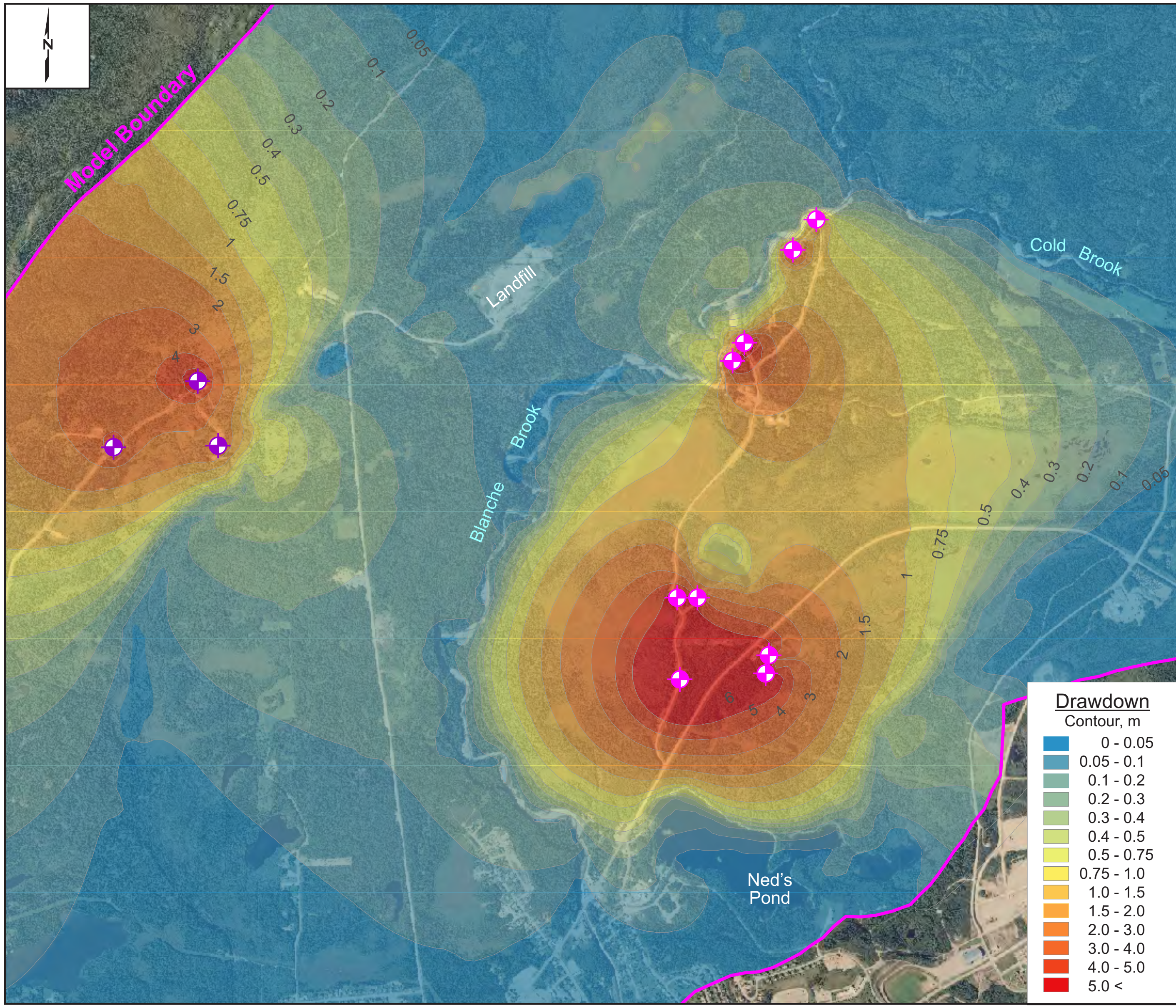
Project No.
555

Location
Stephenville, NL

Document Reference
FFC-NL-555-004

Date
June 2021

FFC




Legend

Production well under pumping condition for simulation - Stephenville

Production well under pumping condition for simulation - Kippens

0 500 1000

Scale, m

Note: Base orthorectified images/maps by Department of Fisheries and Land Resources (2937_Z3, 2938_Z3, and 3038_Z3).

Figure 4.18

Capture area predicted by the model with steady-state drawdown contours in metres for a scenario of pumping nine production wells from the Stephenville well field and three production wells from the Kippens well field with pumping rates shown in Table 4.7.

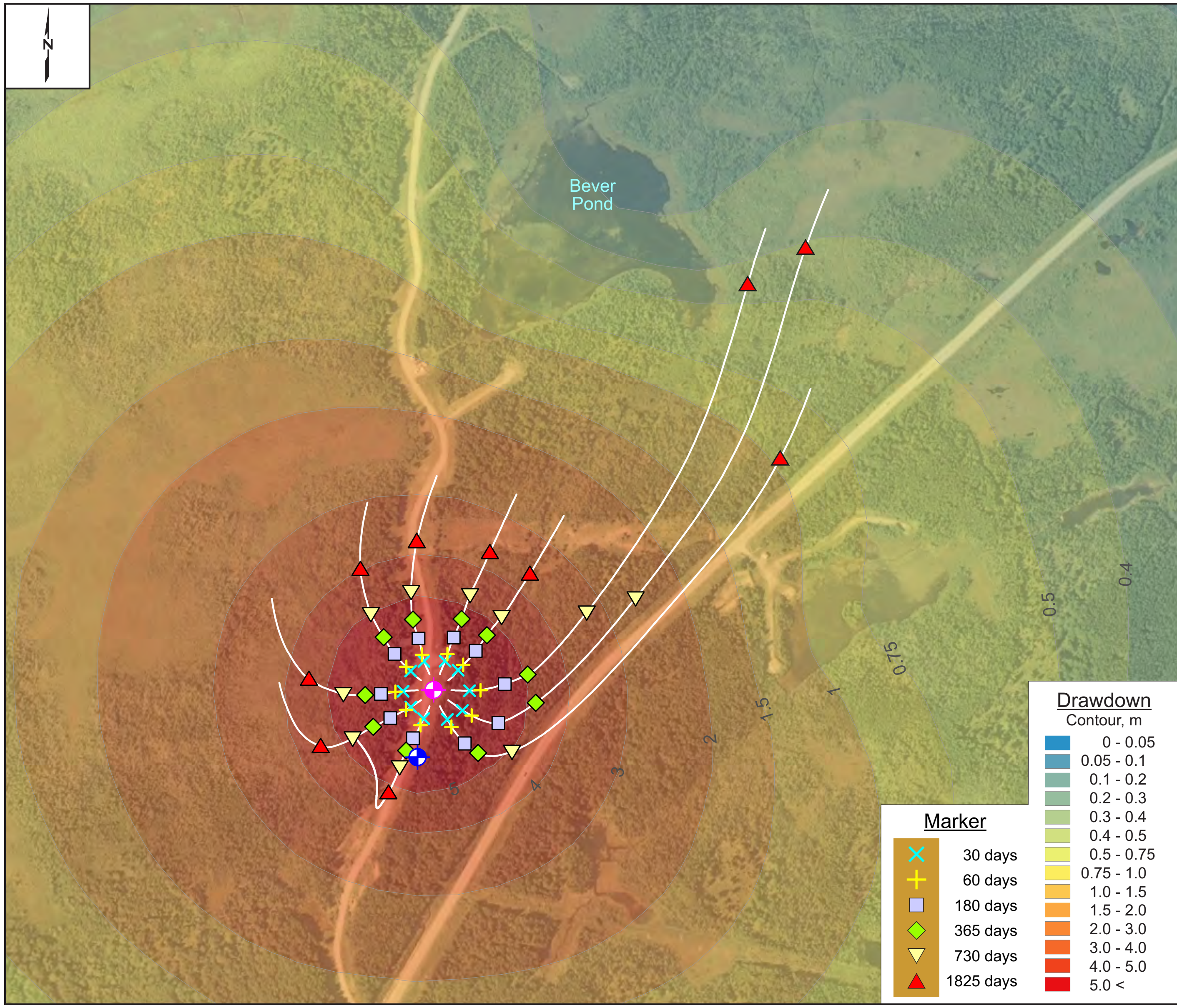
Project No.
555

Location
Stephenville, NL

Document Reference
FFC-NL-555-004

Date
June 2021

FFC




Legend

Production well under pumping condition for simulation

Monitoring well

0 100 200

Scale, m

Note: Base orthorectified images/maps by Department of Fisheries and Land Resources (2937_Z3, 2938_Z3, and 3038_Z3).

Figure 4.19

Reverse particle tracks showing selected pathways with markers for distance travelled between 30 days and 1825 days (5 years) with drawdown contours in metres for a scenario of pumping Well 1 at 1,900 L/min.

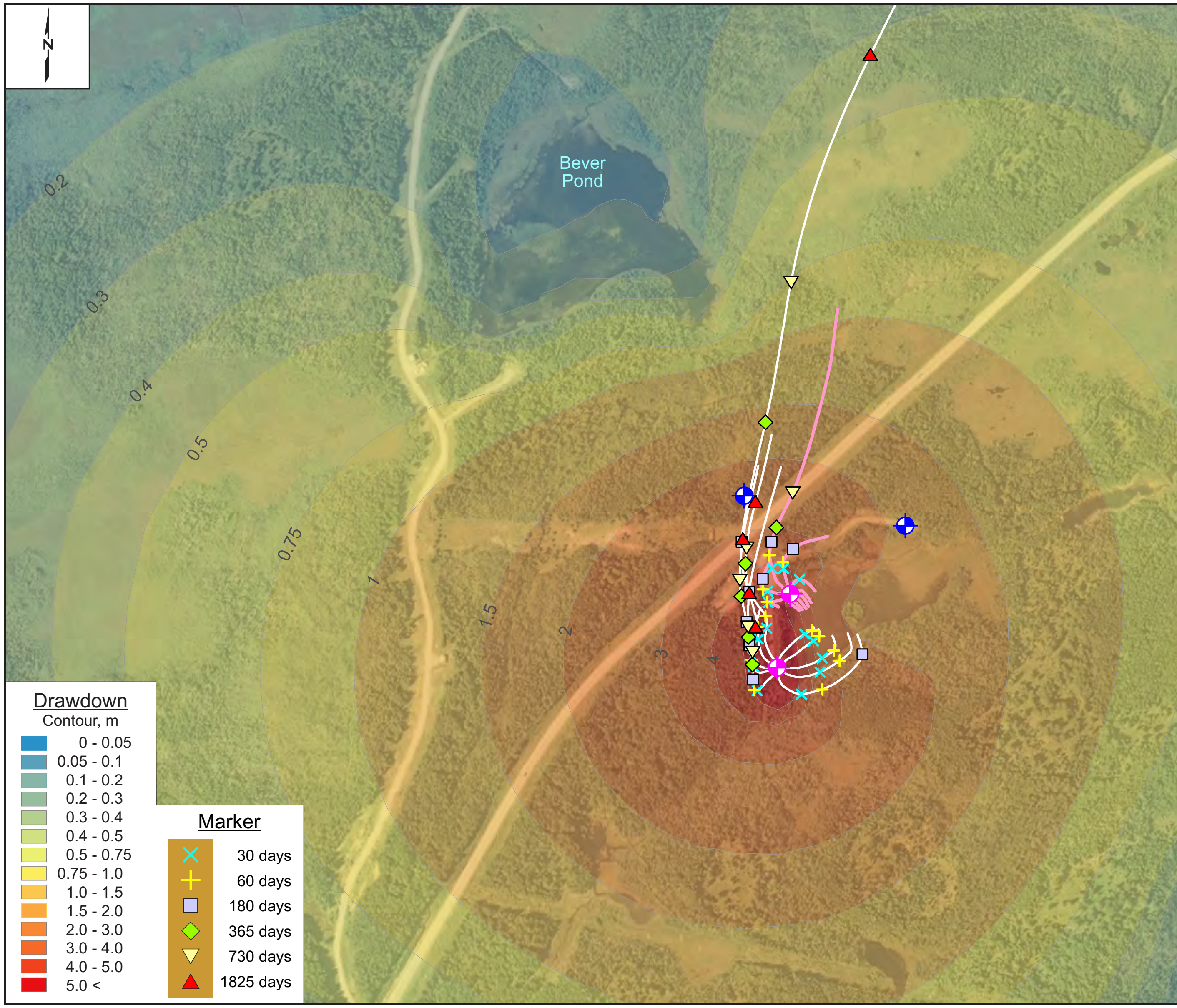
Project No.
555

Location
Stephenville, NL

Document Reference
FFC-NL-555-004

Date
June 2021

FFC




Legend

Production well under pumping condition for simulation

Monitoring well

0 100 200

Scale, m

Note: Base orthorectified images/maps by Department of Fisheries and Land Resources (2937_Z3, 2938_Z3, and 3038_Z3).

Figure 4.20

Reverse particle tracks showing selected pathways with markers for distance travelled between 30 days and 1825 days (5 years) with drawdown contours in metres for a scenario of pumping Well 2 and Well 4 at 800 L/min and 1,200 L/min, respectively.

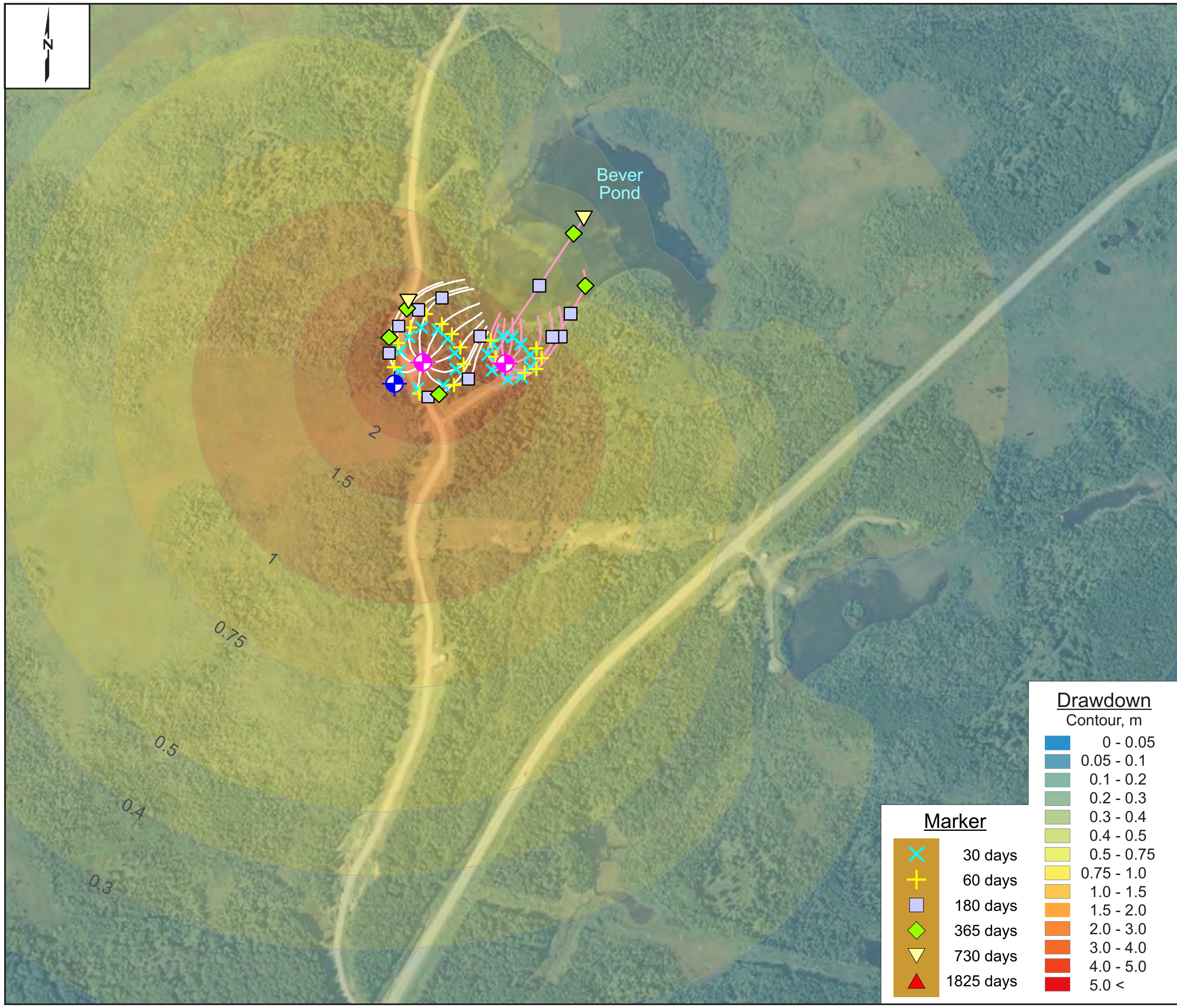
Project No.
555

Location
Stephenville, NL

Document Reference
FFC-NL-555-004

Date
June 2021

FFC




Legend

Production well under pumping condition for simulation

Monitoring well

0 100 200

Scale, m

Note: Base orthorectified images/maps by Department of Fisheries and Land Resources (2937_Z3, 2938_Z3, and 3038_Z3).

Figure 4.21

Reverse particle tracks showing selected pathways with markers for distance travelled between 30 days and 1825 days (5 years) with drawdown contours in metres for a scenario of pumping Well 5 and Well 6 at 700 L/min and 310 L/min, respectively.

Project No.

555

Location

Stephenville, NL

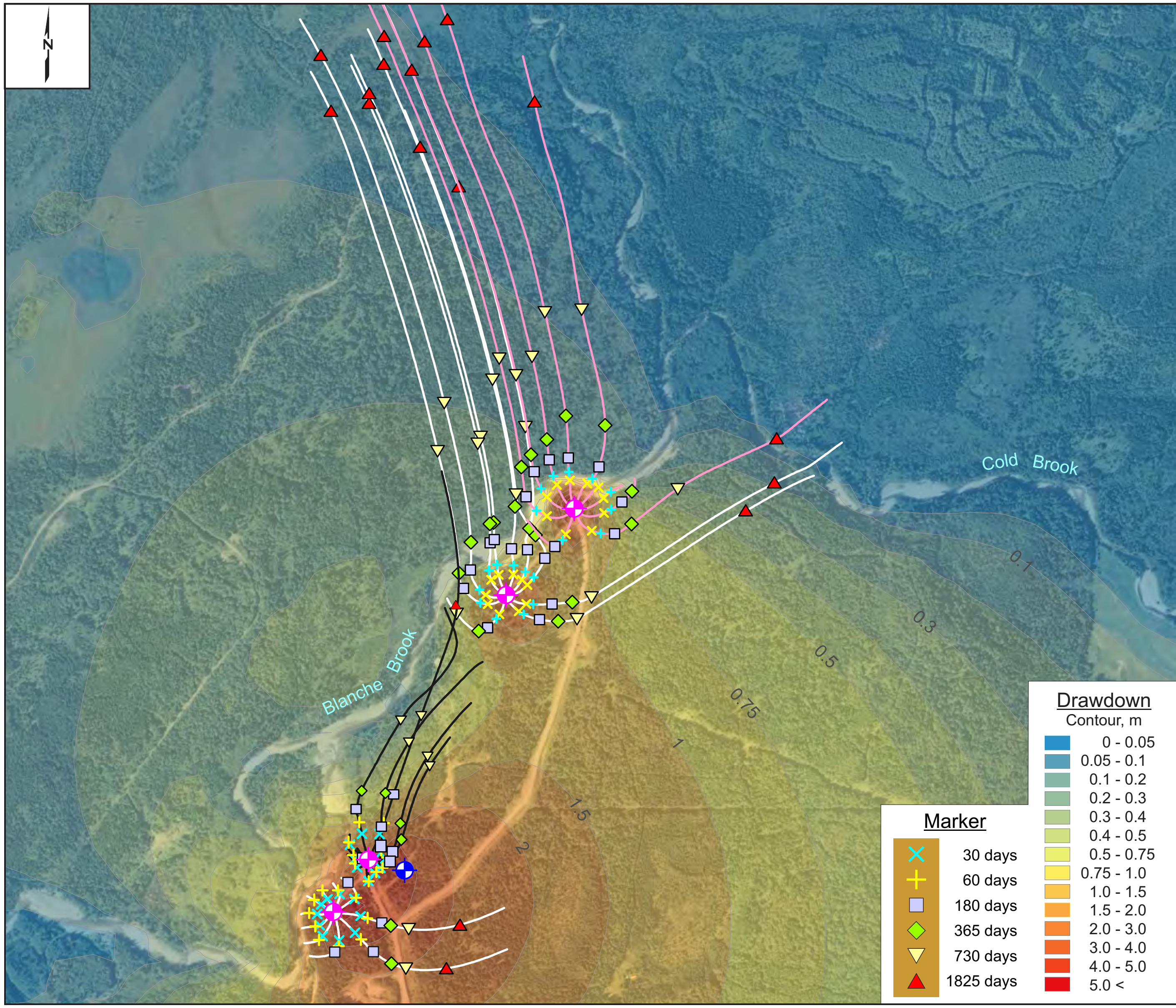
Document Reference

FFC-NL-555-004

Date

June 2021

FFC




Legend

Production well under pumping condition for simulation

Monitoring well

0 100 200 300

Scale, m

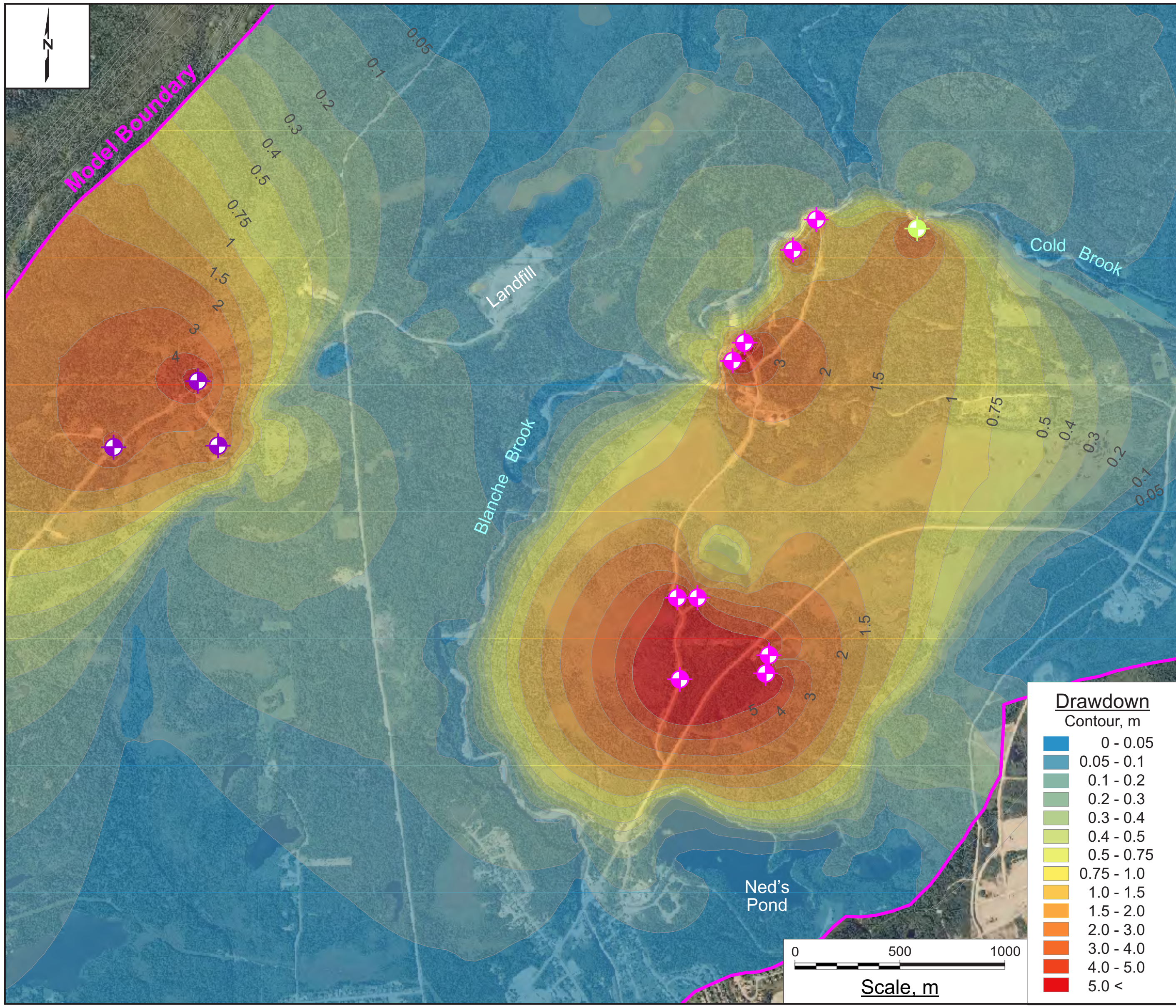
Note: Base orthorectified images/maps by Department of Fisheries and Land Resources (2937_Z3, 2938_Z3, and 3038_Z3).

Figure 4.22

Reverse particle tracks showing selected pathways with markers for distance travelled between 30 days and 1825 days (5 years) with drawdown contours in metres for a scenario of pumping Well 7, Well 8, Well 9 and Well 10 at 1,400, 1,500, 650 and 570 L/min, respectively.

| | |
|---|---|
| Project No. | 555 |
| Location | Stephenville, NL |
| Document Reference | FFC-NL-555-004 |
| Date | June 2021 |

FFC




## Legend

- Production well under pumping condition for simulation - Stephenville
- Production well under pumping condition for simulation - Kippens
- Proposed production well for simulation of additional water supply

Note: Base orthorectified images/maps by Department of Fisheries and Land Resources (2937_Z3, 2938_Z3, and 3038_Z3).

Figure 4.23

Capture area predicted by the model with steady-state drawdown contours in metres for a scenario of pumping one proposed well, FW2 as well as twelve existing wells from both Stephenville and Kippens well fields with pumping rates shown in Table 4.7.

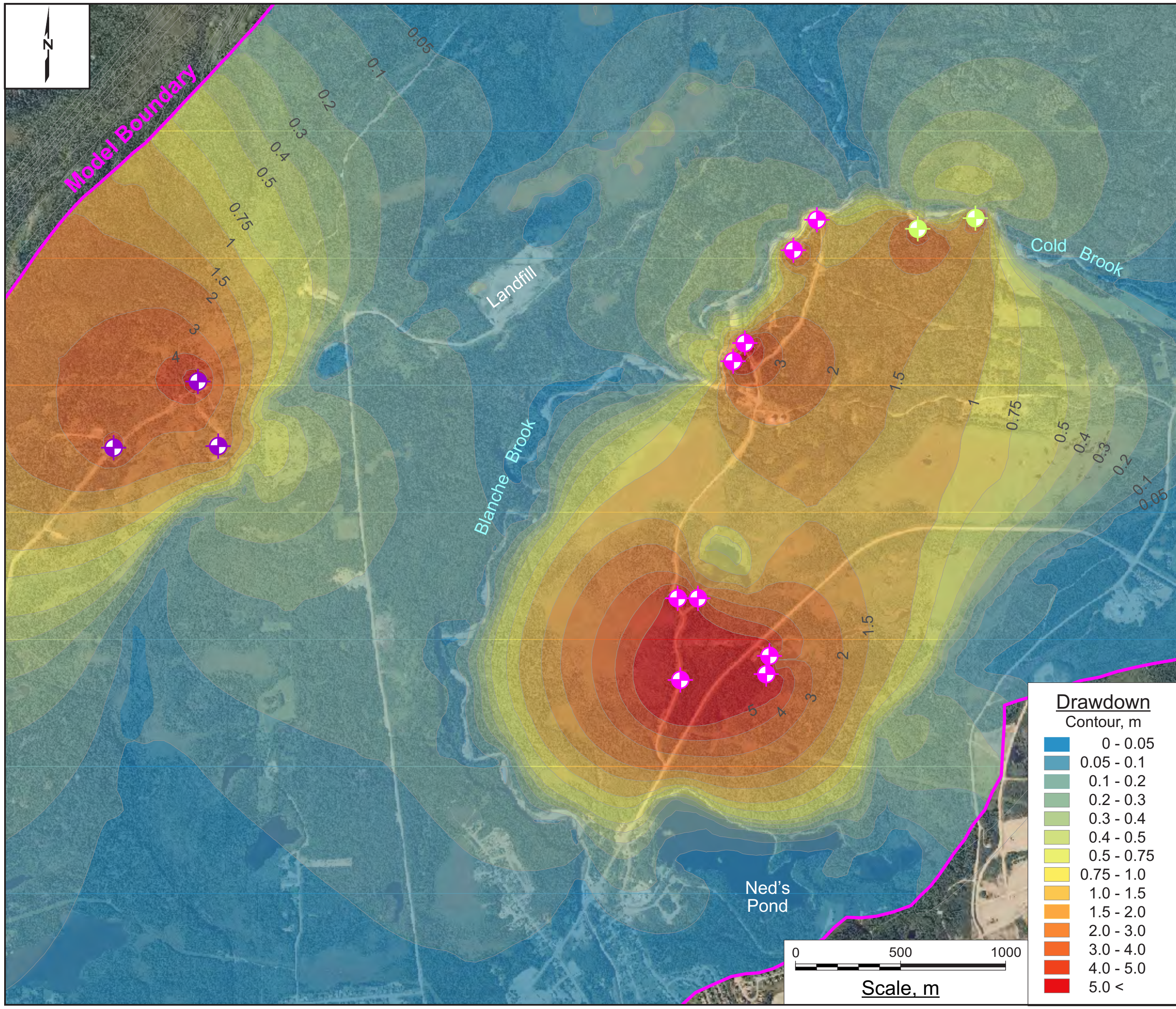
Project No.
555

Location
Stephenville, NL

Document Reference
FFC-NL-555-004

Date
June 2021

FFC




## Legend

- Production well under pumping condition for simulation - Stephenville
- Production well under pumping condition for simulation - Kippens
- Proposed production well for simulation of additional water supply

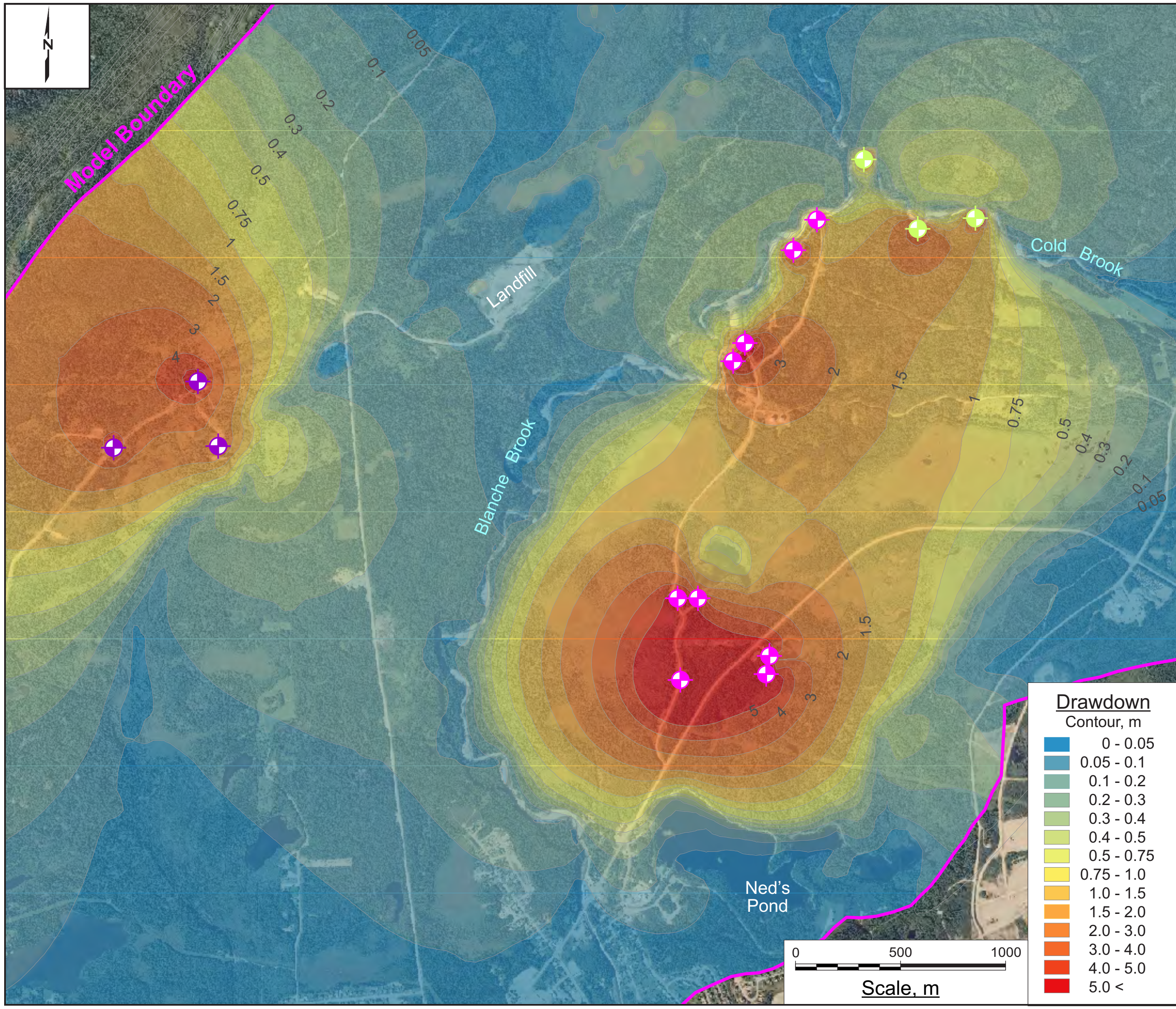
Note: Base orthorectified images/maps by Department of Fisheries and Land Resources (2937_Z3, 2938_Z3, and 3038_Z3).

Figure 4.24

Capture area predicted by the model with steady-state drawdown contours in metres for a scenario of pumping two proposed wells, FW2 and FW3 as well as 12 wells from both Stephenville and Kippens well fields with pumping rates shown in Table 4.7.

| | |
|---|---|
| Project No. | 555 |
| Location | Stephenville, NL |
| Document Reference | FFC-NL-555-004 |
| Date | June 2021 |

FFC




**Legend**

- Production well under pumping condition for simulation - Stephenville
- Production well under pumping condition for simulation - Kippens
- Proposed production well for simulation of additional water supply

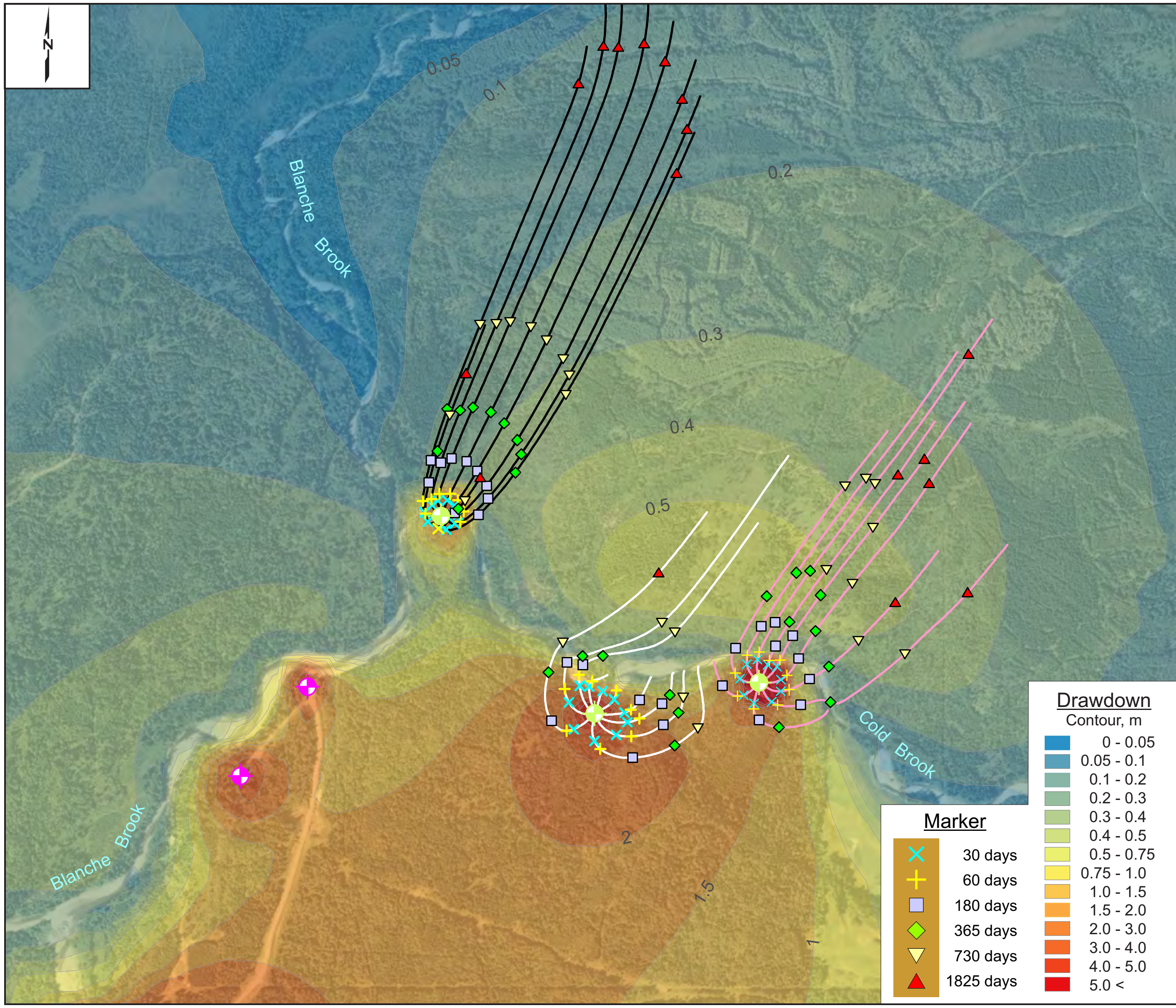
Note: Base orthorectified images/maps by Department of Fisheries and Land Resources (2937_Z3, 2938_Z3, and 3038_Z3).

Figure 4.25

Capture area predicted by the model with steady-state drawdown contours in metres for a scenario of pumping three proposed wells as well as 12 existing wells from both Stephenville and Kippens well fields with pumping rates shown in Table 4.7.

| | |
|---|---|
| Project No. | 555 |
| Location | Stephenville, NL |
| Document Reference | FFC-NL-555-004 |
| Date | June 2021 |

FFC




## Legend

Proposed well

Production/Pumping Well

0 100 200 300

Scale, m

Note: Base orthorectified images/maps by Department of Fisheries and Land Resources (2937_Z3, 2938_Z3, and 3038_Z3).

Figure 4.26

Reverse particle tracks showing selected pathways with markers for distance travelled between 30 days and 1825 days (5 years) with drawdown contours in metres for a scenario of pumping three proposed wells at 1,000 L/min each as well as twelve existing wells.

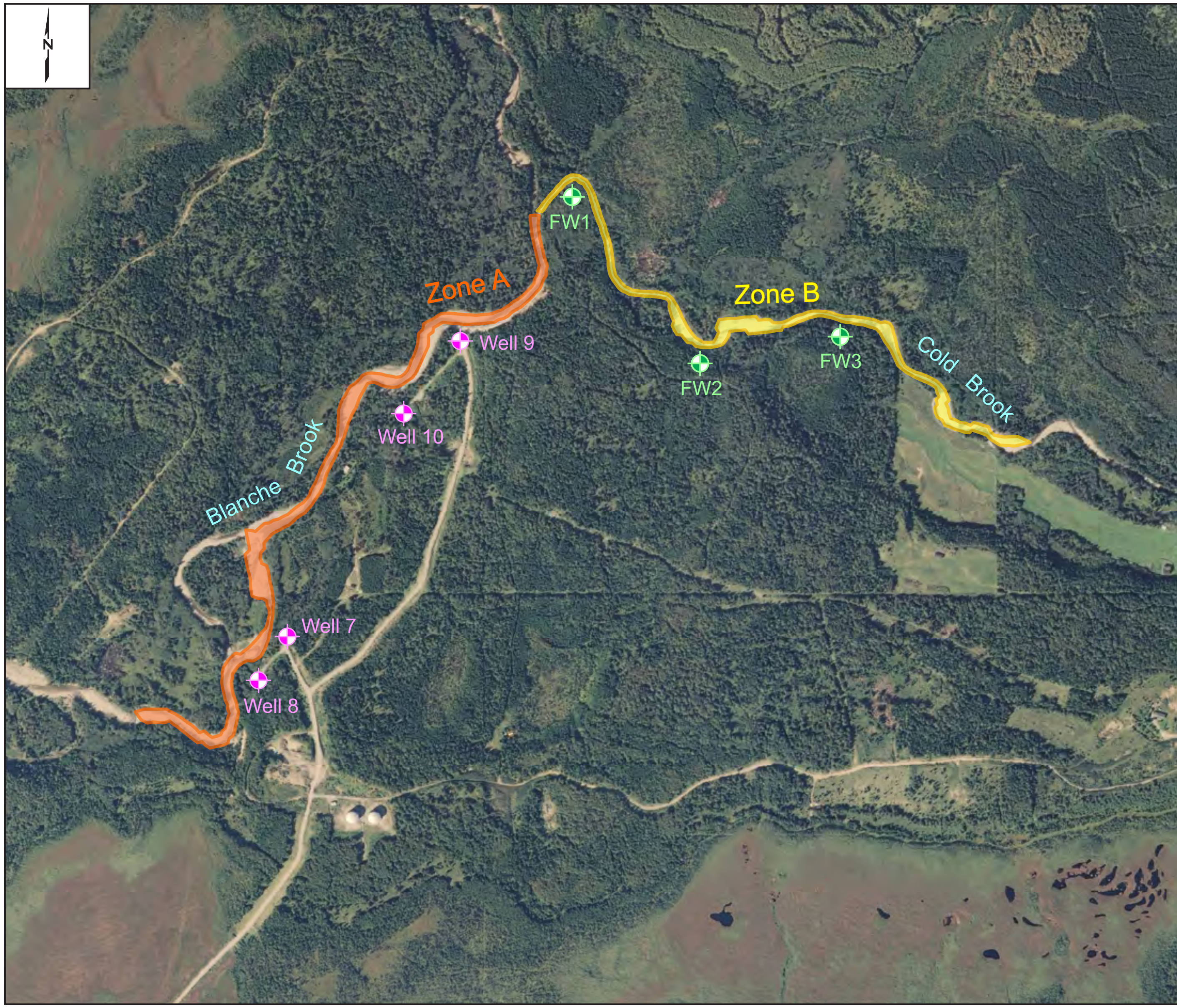
Project No.
555

Location
Stephenville, NL

Document Reference
FFC-NL-555-004

Date
June 2021

FFC




Legend

Production well

Proposed well

0 100 200 300 400

Scale, m

Note: Base orthorectified images/maps by Department of Fisheries and Land Resources (2937_Z3, 2938_Z3, and 3038_Z3).

Figure 4.27

Two zones along Blanche Brook and Cold Brook for flux calculation to assess impact of well field production on baseflow.

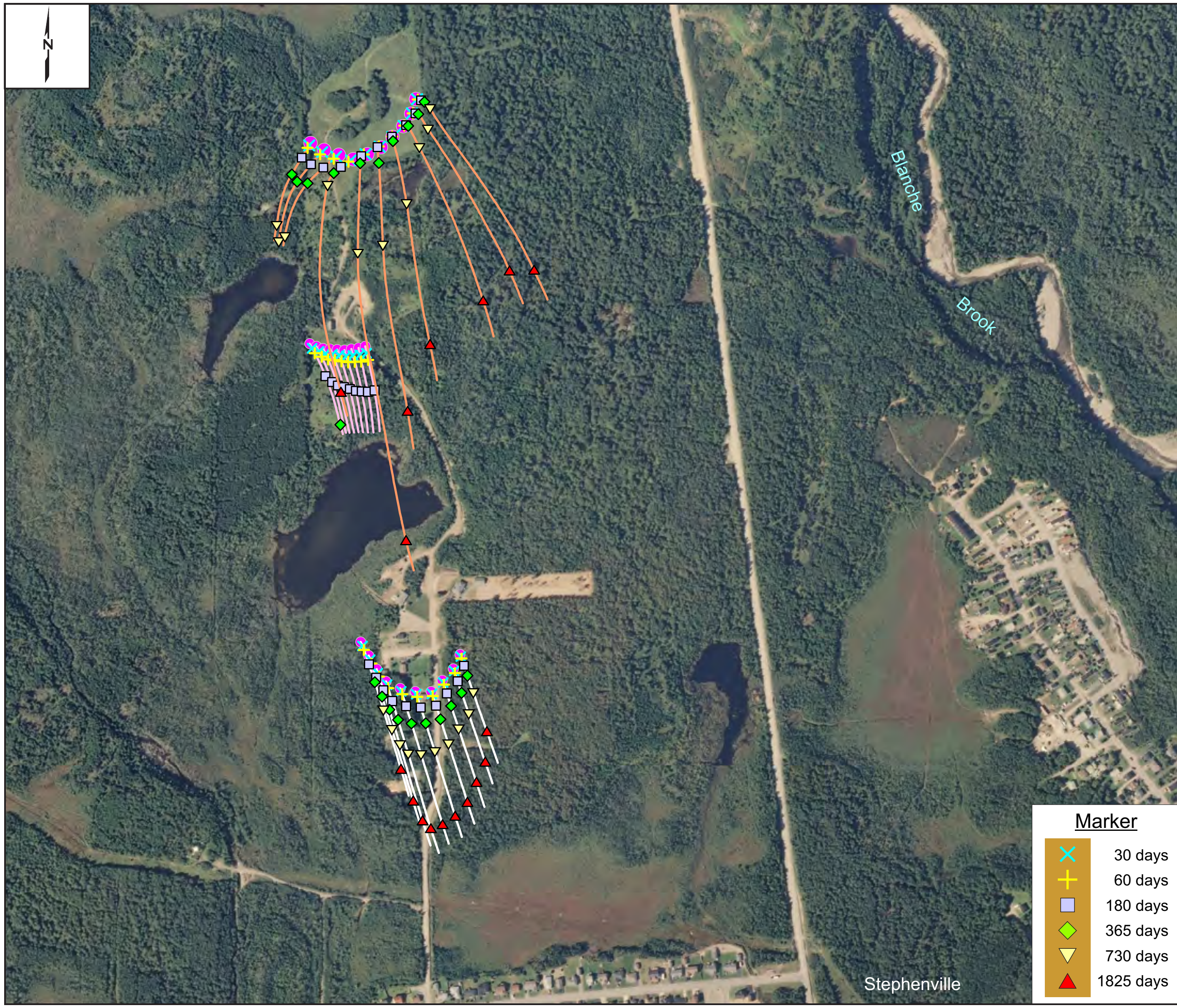
Project No. 555

Location Stephenville, NL

Document Reference FFC-NL-555-004

Date June 2021

FFC




Legend

Particle Release Point

0 100 200 300

Scale, m

Note: Base orthorectified images/maps by Department of Fisheries and Land Resources (2937_Z3, 2938_Z3, and 3038_Z3).

Figure 4.28

Forward particle tracks showing selected pathways with markers for distance travelled between 30 days and 1825 days (5 years) around residential areas at the north side of the Town of Stephenville.

Project No.
555

Location
Stephenville, NL

Document Reference
FFC-NL-555-004

Date
June 2021

FFC

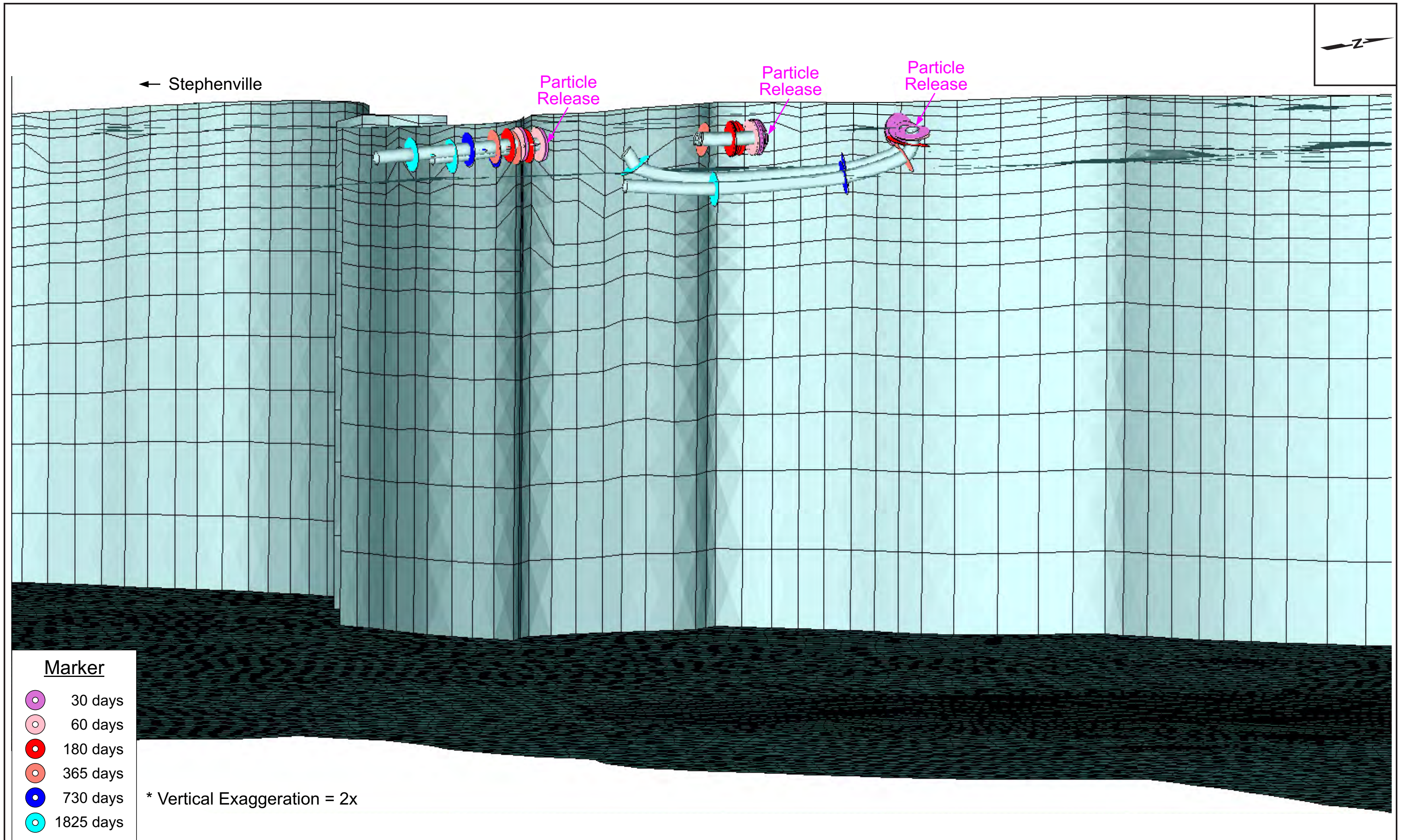



Figure 4.29 Cross-sectional view of the forward particle tracks and pathways shown on the plan view in Figure 4.28.

| Project No. | Document Reference |
|---|---|
| 555 | FFC-NL-555-004 |
| **Location** | **Date** |
| Stephenville, NL | June 2021 |

FFC

# 5.0 OPTIONS FOR MEETING EXISTING AND FUTURE WATER DEMANDS

## 5.1 Well Field Maintenance

The Stephenville production wells, except for the main bedrock well - Well 1, were constructed using a K-Packer assembly. For wells with a K-Packer construction, a gravel pack cannot be installed around the well screen and a natural gravel/filter pack is developed by both the normal well development procedures and by long term pumping with repeated on and off cycles in each well. The object is to remove the fines from the aquifer that is in immediate contact with the well screen, to maintain permeability and reduce well loss. However, over time with sustained pumping the well screen becomes partially clogged by fines and in some cases by bacterial growth on the well screen producing a reduction in specific capacity and loss of well yield.

Well field maintenance requires that screened wells be inspected on a regular basis by calculating the current specific capacity of each well to identify those wells where the specific capacity is lower than the well's original specific capacity. In addition, the aquifer capacity has to be evaluated by measuring the static water level in each well to identify those wells in which the well withdrawals are exceeding the local aquifer capacity, taking into account any well interference impacts on water levels from any nearby active production wells. Note that during the original aquifer tests, the drawdowns or well interference in nearby wells were measured and recorded.

For wells that show a reduction in specific capacity, but no significant reduction in static water levels, a Biological Activation Reaction Tests (BARTs) test should be performed. If the BARTs test does not show any obvious bacterial growth in the well, it is reasonable to assume that the reduction in specific capacity is due to an accumulation of fines around the well screen. To remove the fines from around the well screen, an aggressive program of well re-development using simultaneous surging with a surge block and air-lifting to remove the accumulated fines needs to be undertaken on a regular basis followed by measurement of the specific capacity of each well. Restoring or improving the specific capacity of productions wells, in the absence of major reductions in static water levels, is the most cost effective approach to increasing or restoring overall water supply.

To determine the current static water level for existing wells, each well has to be shut down for a 24-hour to 48-hour period with continuous monitoring of the water levels, with a five-minute measurement interval, in each shut-in well and in each nearby pumping well and observation well. The goal is to determine if the aquifer capacity is being exceeded by excessive long term aquifer withdrawals. These data will inform the Town of the need for additional wells and provide guidance on where any new production wells are best located.

It is recommended that any new production wells should be constructed, once the aquifer geology has been established, by diamond drilling with packer testing at the proposed production well location, by first driving a 300 mm (12-inch) casing to bedrock or to the planned well depth, then assembling and placing a 200 mm (8-inch) well screen and casing assembly in the 300 mm (12-inch) casing, followed by placing a silica sand filter pack around the well screen as the 300 mm (12-inch) casing is withdrawn with placement of a standard bentonite well seal and concrete collar above the well screen and at the top of the well. Once the well has been constructed, the normal sequence of well development using surging and air-lifting needs to be completed to settle the silica sand filter pack. This design, using an artificial filter pack, while more expensive to construct, is expected to reduce the frequency of well re-development and overall well maintenance.

## 5.2 Additional Production Well Locations

The 3D model simulations show that the Town cannot develop additional bedrock wells in the area at the northeast end of Beaver Pond. Also, the model simulations and the current static water levels suggest that this bedrock aquifer is being over exploited. This is most likely due to the addition of a fourth well in the Kippens' well field without any obvious or known assessment of the long-term impact on the common bedrock aquifer.

The remaining options include constructing one or more production wells at and east of the point where the Cold Brook tributary joins Blanche Brook. **Figure 4.10** shows three potential locations and the expected drawdowns for well withdrawals of 1,000 L/min (264 USgpm). Construction of one or more wells will require a site investigation at each location to determine how close the wells can be placed to the Cold Brook tributary or to Blanche Brook.

Additional production wells can be constructed east of this location along the edge of the farm property adjacent to the Cold Brook tributary but each production well will impact, to some extent, the baseflow conditions in the Cold Brook tributary and hence in Blanche Brook.

The other major aquifer in the Stephenville area is the high capacity granular aquifer which currently provides the water supply for the NHSL fish hatchery. This aquifer is located under Warm Creek and to the east of Warm Creek. Wells developed in this aquifer have the capacity to produce approximately 1,890 L/min (500 USgpm). A production well that is developed on the west side of Warm Creek will have limited impact on the overall yield from this granular aquifer.

# 6.0 REFERENCES

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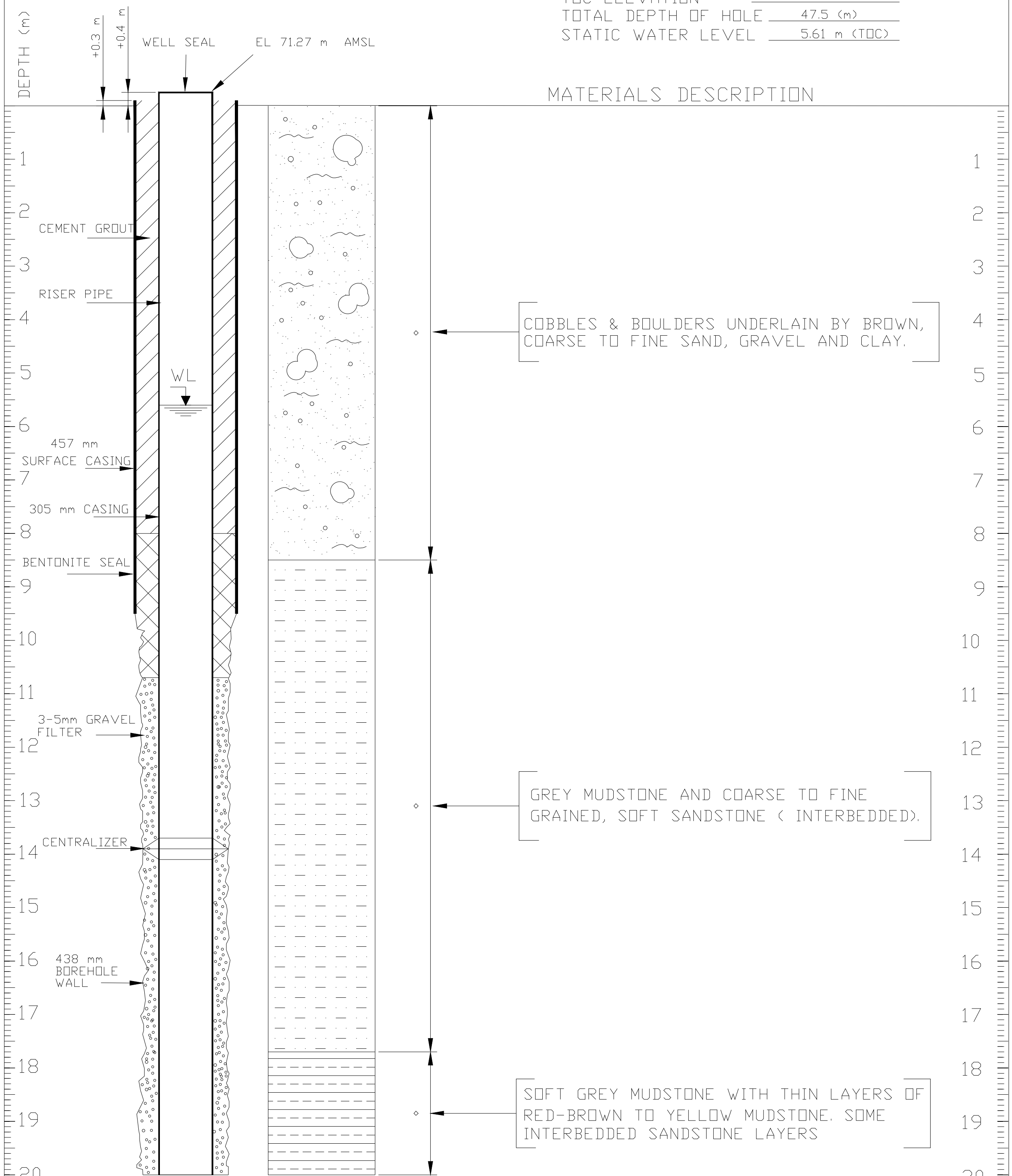
Williams, H., 1985. Bedrock Geology, Geological Survey of Canada, Map (1579A).

# *APPENDIX A*

## ***Production Well Construction Logs***

# BORING BH- PTW-1

CLIENT NAME STEPHENVILLE
DATE DRILLED DEC 1--2 / 1997
DATE REAMED: DEC 1--2 / 1998
TOC ELEVATION 71.27 (m)
TOTAL DEPTH OF HOLE 47.5 (m)
STATIC WATER LEVEL 5.61 m (TOC)

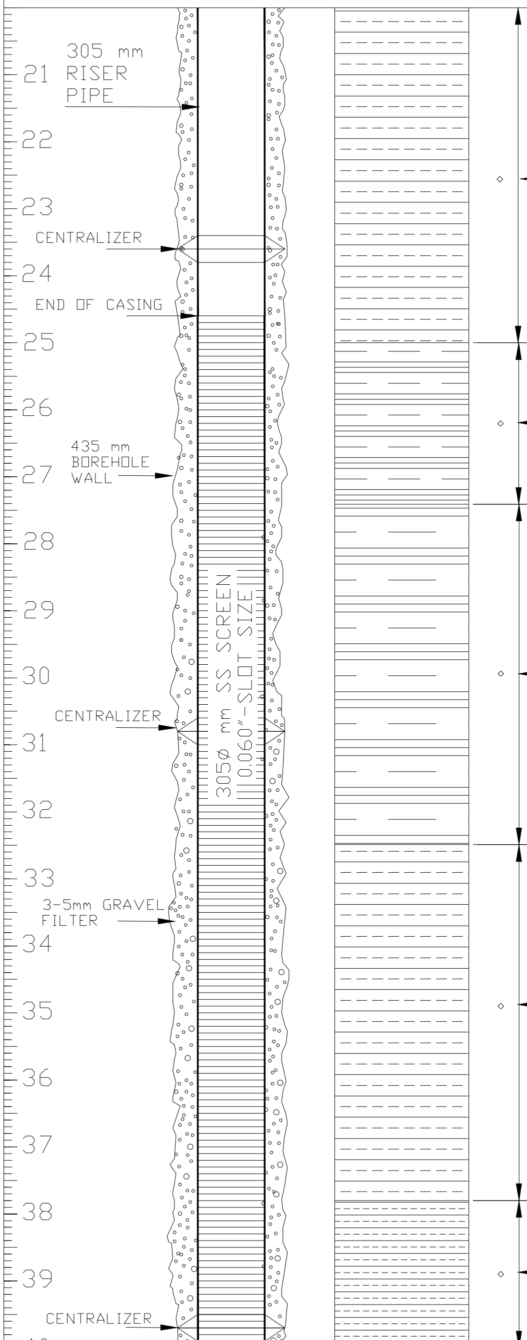


# BORING BH- PTW-1

CLIENT NAME: STEPHENVILLE
DATE DRILLED: DEC 1--2 / 1997
DATE REAMED: DEC 1--2 / 1998
TOC ELEVATION: 71.27 (m)
TOTAL DEPTH OF HOLE: 47.5 (m)
STATIC WATER LEVEL: 5.61 m (TOC)

DEPTH (m)

MATERIALS DESCRIPTION




SOFT GREY MUDSTONE WITH THIN LAYERS OF RED-BROWN TO YELLOW MUDSTONE. SOME INTERBEDDED SANDSTONE LAYERS.

HARD, GREY, FINE GRAINED SANDSTONE.

POORLY CEMENTED, FRIABLE, COARSE GRAINED SANDSTONE WITH PYRITE. (WATER INCREASE)

HARD, BROWN TO GREY, COARSE SANDSTONE WITH INTERBEDDED LAYERS OF DARK-GREY SILTSTONE.

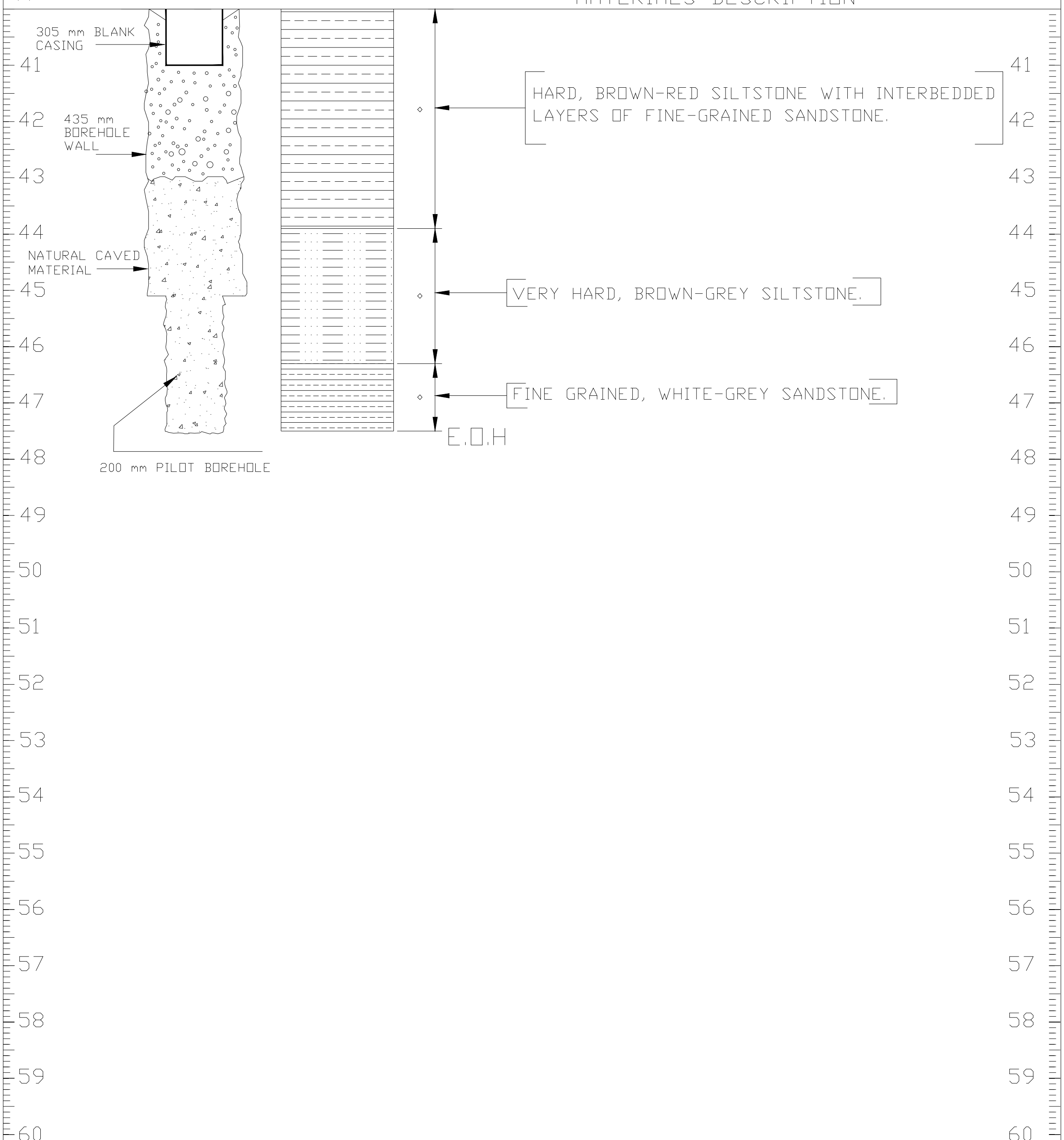
HARD, BROWN-RED SILTSTONE WITH INTERBEDDED LAYERS OF FINE-GRAINED SANDSTONE.

# BORING BH- PTW-1

CLIENT NAME: STEPHENVILLE
DATE DRILLED: DEC 1--2 / 1997
DATE REAMED: DEC 1--2 / 1998
TOC ELEVATION: 71.27 (m)
TOTAL DEPTH OF HOLE: 47.5 (m)
STATIC WATER LEVEL: 5.61 m (TOC)

DEPTH (m)

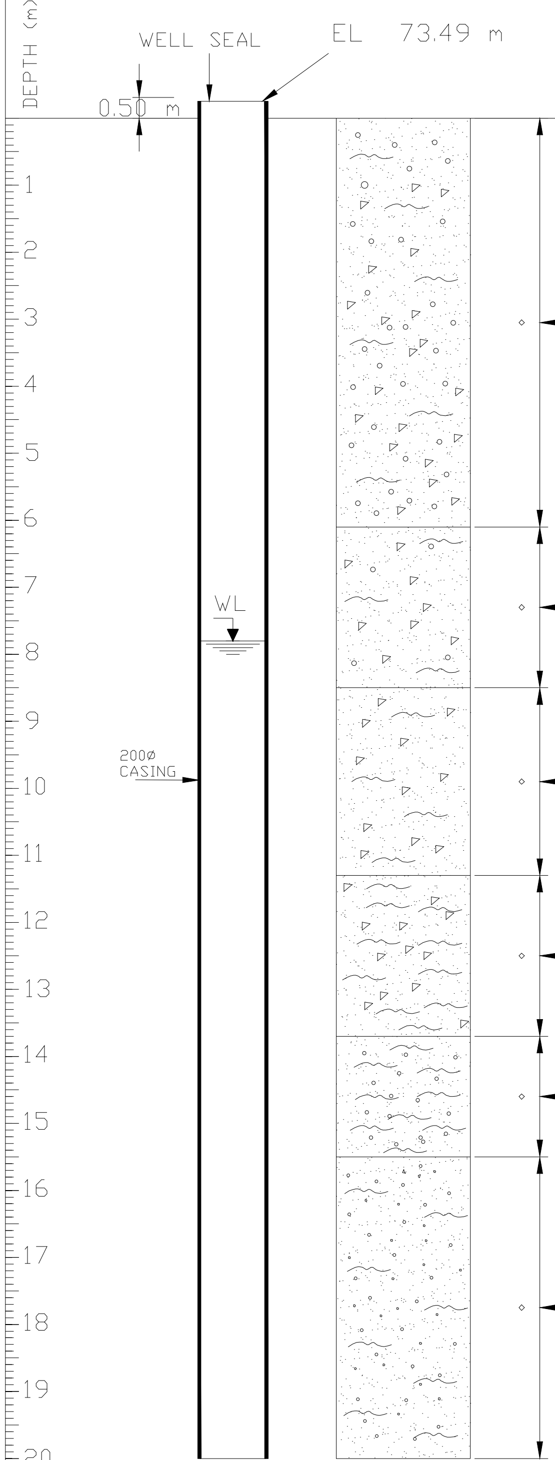
MATERIALS DESCRIPTION




FRACFLOW CONSULTANTS INC.

# BORING BH- PTW-2

CLIENT NAME: STEPHENVILLE
DATE DRILLED: DEC 6-7 / 1997
TOC ELEVATION: 73.49 m
TOTAL DEPTH OF HOLE: 34.70 m
STATIC WATER LEVEL: 7.92 m (TOC)



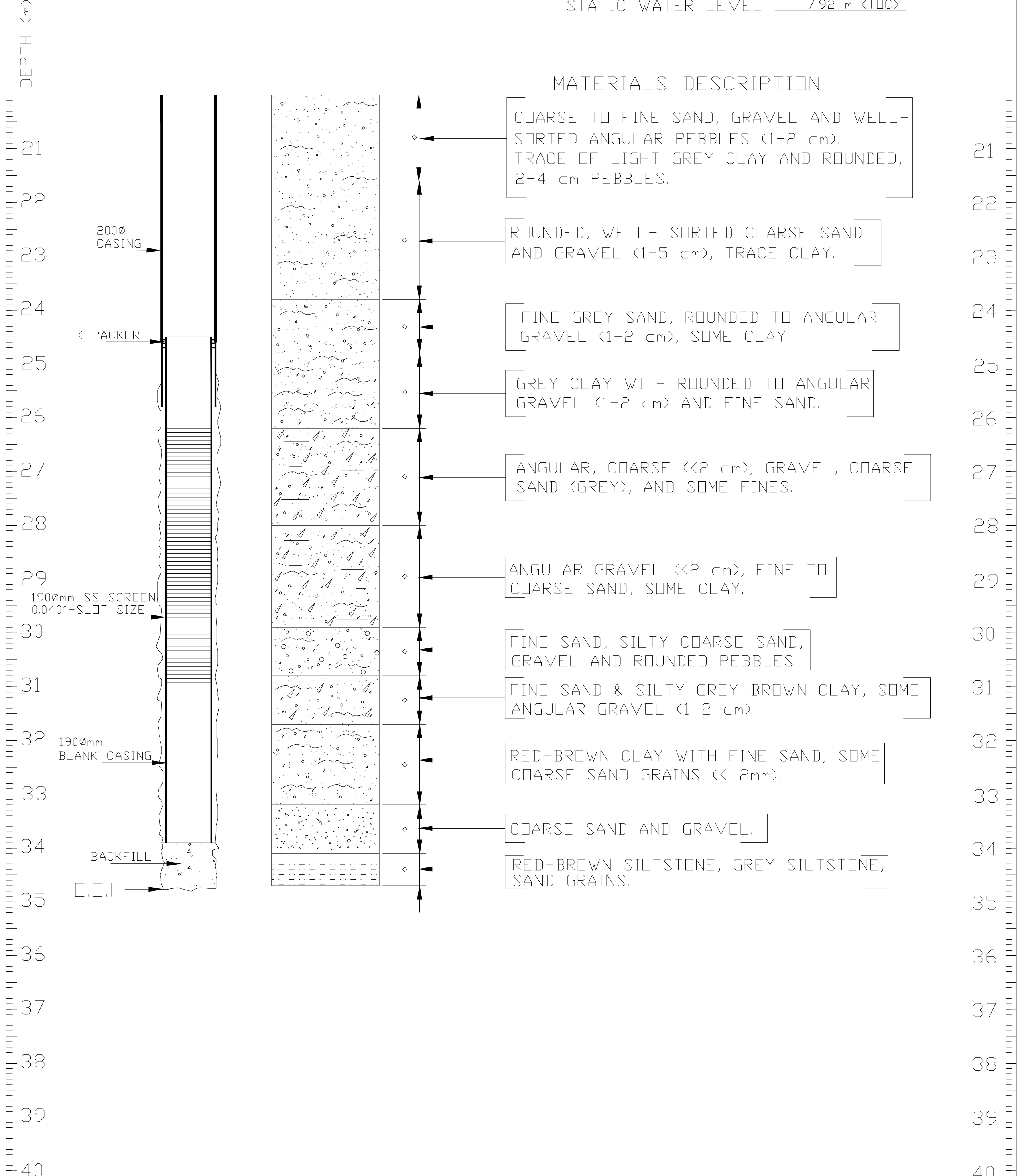

MATERIALS DESCRIPTION

| Depth (m) | Description |
|---|---|
| 0 – 6 | COBBLES UNDERLAIN BY BROWN, COARSE SAND, GRAVEL AND ROUNDED TO ANGULAR PEBBLES ( 5 cm INITIALLY, <3 cm IN LOWER STRATA. ) |
| 6 – 8.4 | COARSE SAND AND FINE ANGULAR GRAVEL (2-6 mm) WITH CLAY; LIGHT BROWN. |
| 8.4 – 11.2 | FINE BROWN SAND AND CLAY; ANGULAR GRAVEL (1-2 cm). |
| 11.2 – 13.6 | BROWN CLAY WITH FINE SAND AND POORLY SORTED ANGULAR GRAVEL (2-4 cm). |
| 13.6 – 15.4 | LIGHT BROWN CLAY WITH ROUNDED GRAVEL AND FINE SAND. |
| 15.4 – 20 | COARSE TO FINE SAND, GRAVEL AND WELL-SORTED ANGULAR PEBBLES (1-2 cm). TRACE OF LIGHT GREY CLAY AND ROUNDED, 2-4 cm PEBBLES. |

FRACFLOW CONSULTANTS INC.

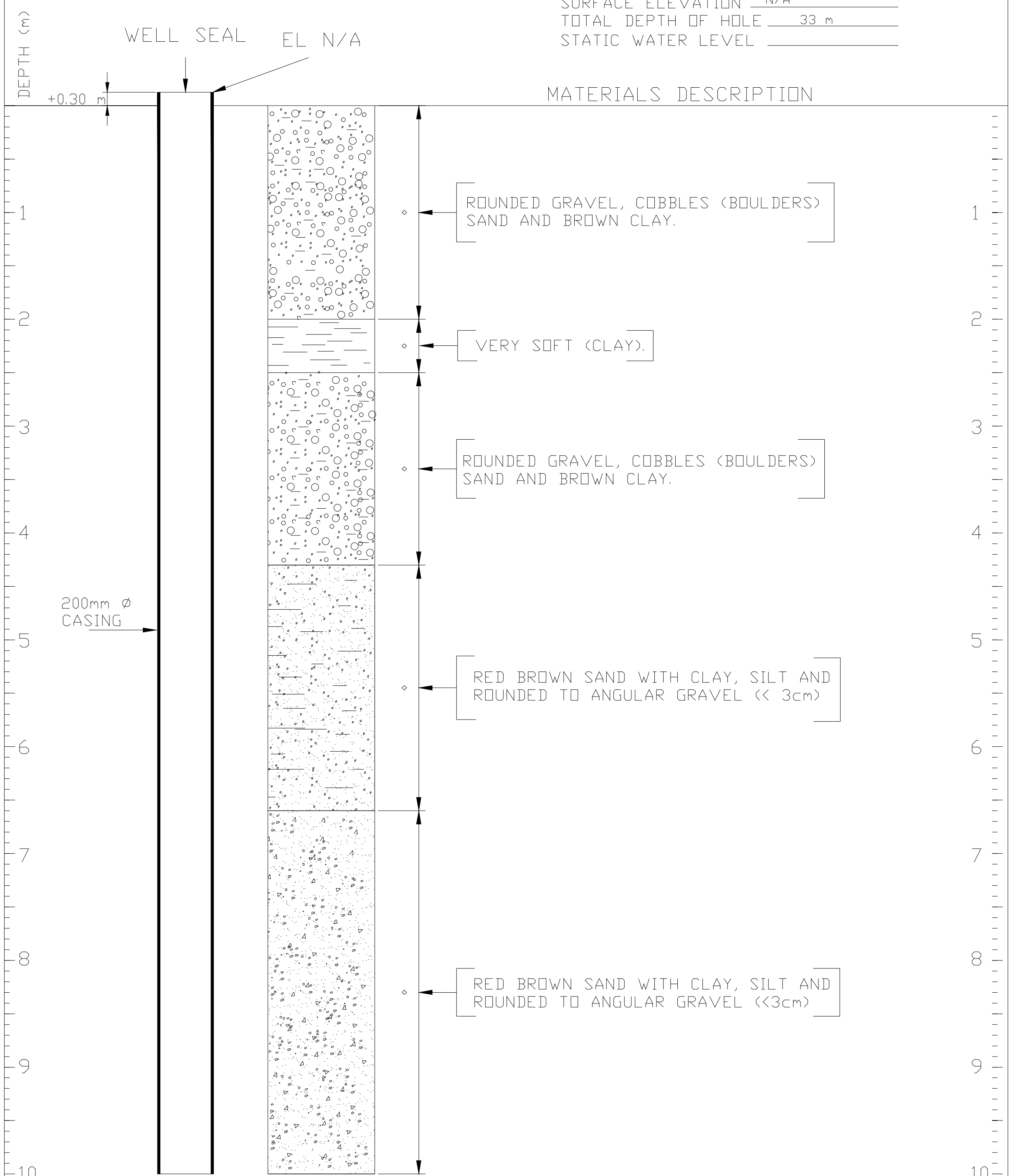
# BORING BH- PTW-2

CLIENT NAME STEPHENVILLE
DATE DRILLED DEC 6-7 / 1997
TOC ELEVATION 73.49 m
TOTAL DEPTH OF HOLE 34.70 m
STATIC WATER LEVEL 7.92 m (TOC)



# BORING BH- PTW-3

CLIENT NAME STEPHENVILLE
DATE DRILLED DEC 5,6/98
SURFACE ELEVATION N/A
TOTAL DEPTH OF HOLE 33 m
STATIC WATER LEVEL




MATERIALS DESCRIPTION

ROUNDED GRAVEL, COBBLES (BOULDERS) SAND AND BROWN CLAY.

VERY SOFT (CLAY).

ROUNDED GRAVEL, COBBLES (BOULDERS) SAND AND BROWN CLAY.

RED BROWN SAND WITH CLAY, SILT AND ROUNDED TO ANGULAR GRAVEL (< 3cm)

RED BROWN SAND WITH CLAY, SILT AND ROUNDED TO ANGULAR GRAVEL (<3cm)

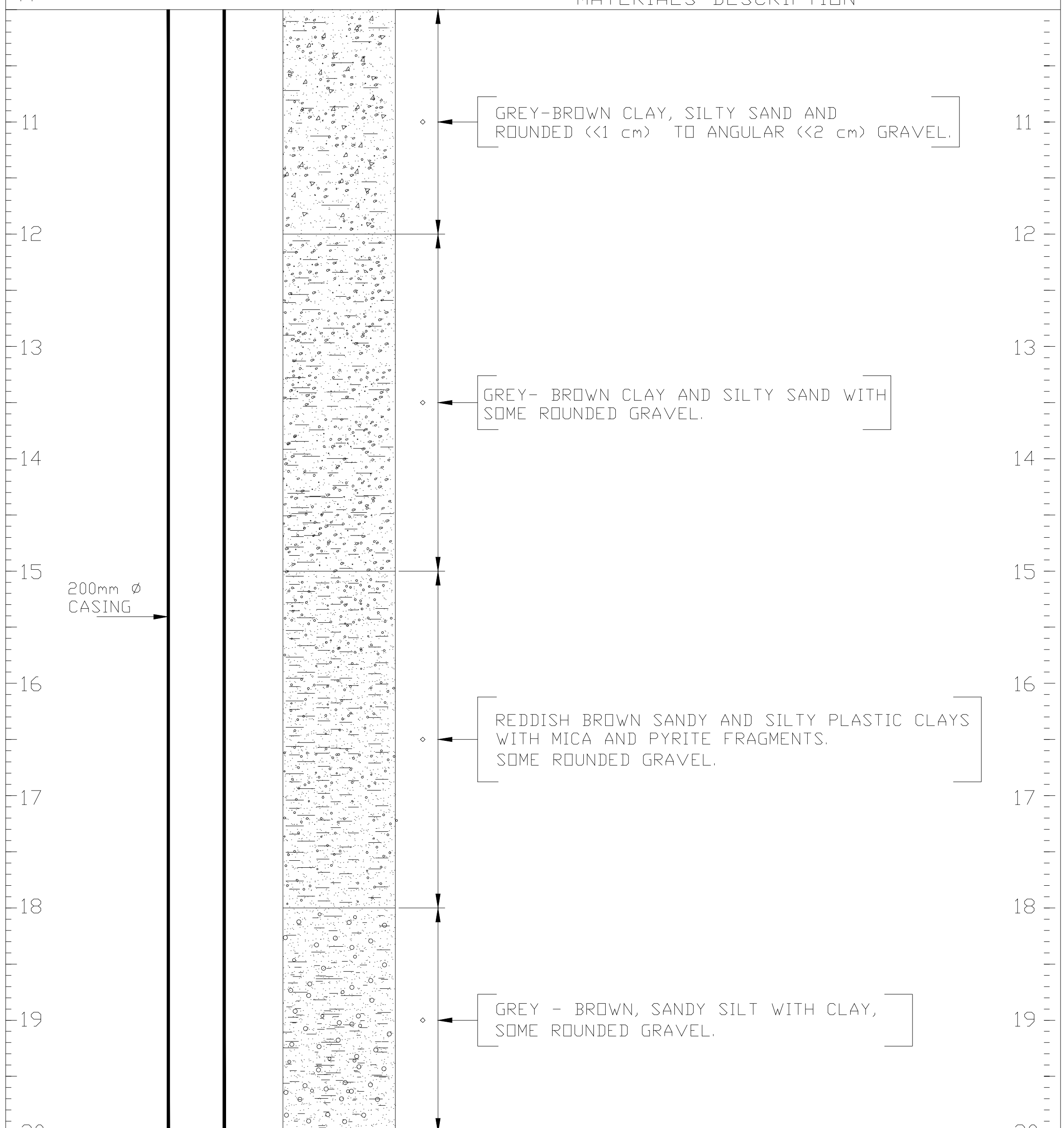
FRACFLOW CONSULTANTS INC.

# BORING BH- PTW-3

CLIENT NAME STEPHENVILLE
DATE DRILLED DEC 5,6/98
SURFACE ELEVATION N/A
TOTAL DEPTH OF HOLE 33 m
STATIC WATER LEVEL

DEPTH (m)

MATERIALS DESCRIPTION



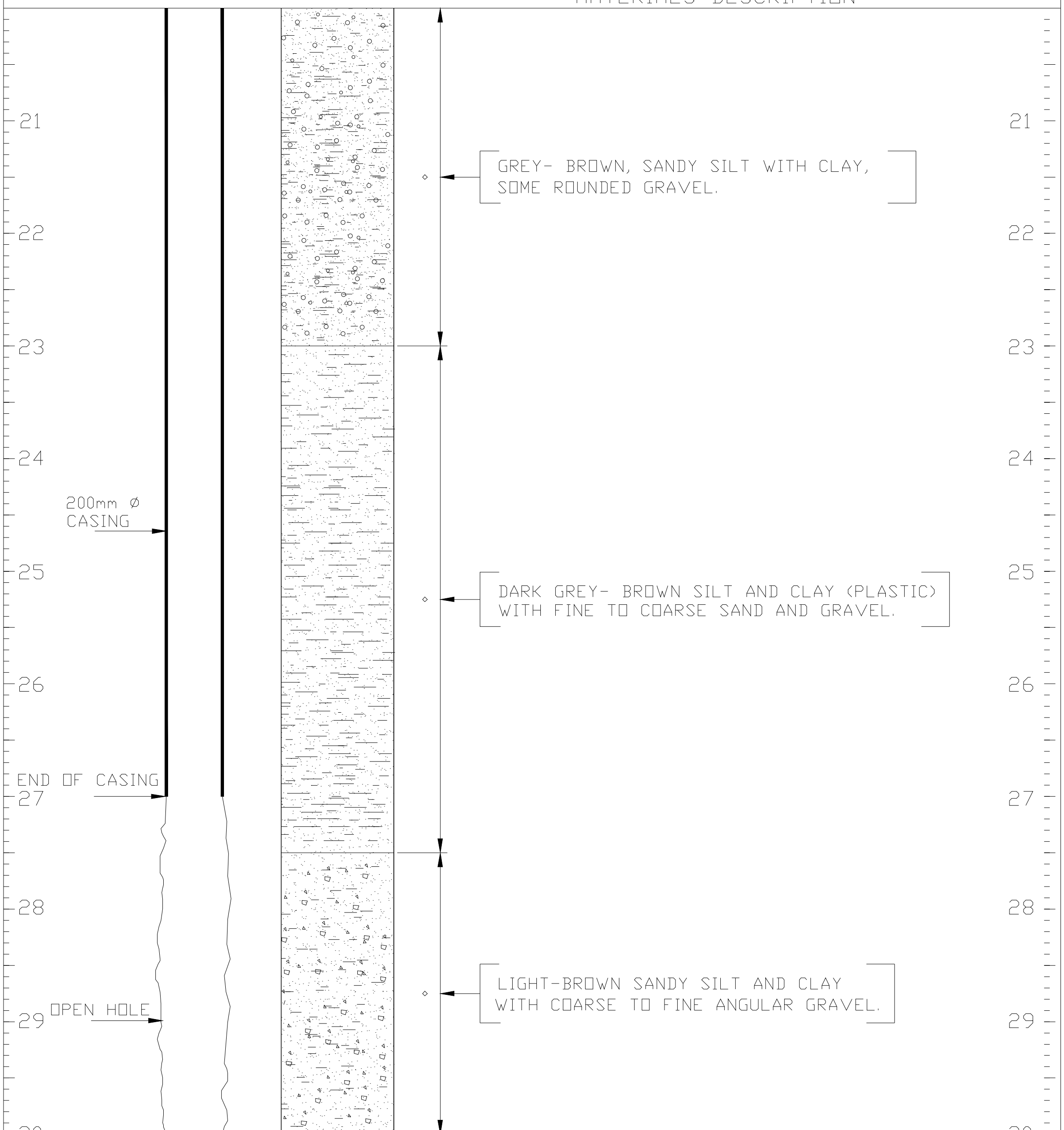

FRACFLOW CONSULTANTS INC.

# BORING BH- PTW-3

CLIENT NAME STEPHENVILLE
DATE DRILLED DEC 5,6/98
SURFACE ELEVATION N/A
TOTAL DEPTH OF HOLE 33 m
STATIC WATER LEVEL

DEPTH (m)

MATERIALS DESCRIPTION

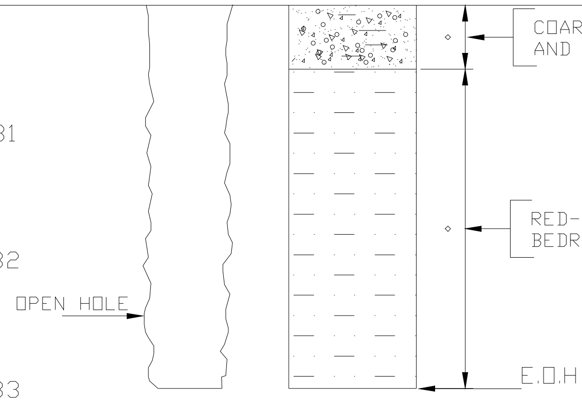


# BORING BH- PTW-3

CLIENT NAME STEPHENVILLE
DATE DRILLED DEC 5,6/98
SURFACE ELEVATION N/A
TOTAL DEPTH OF HOLE 33 m
STATIC WATER LEVEL

DEPTH (m)

MATERIALS DESCRIPTION




COARSE SAND AND GRAVEL WITH RED-BROWN SILT AND CLAY.

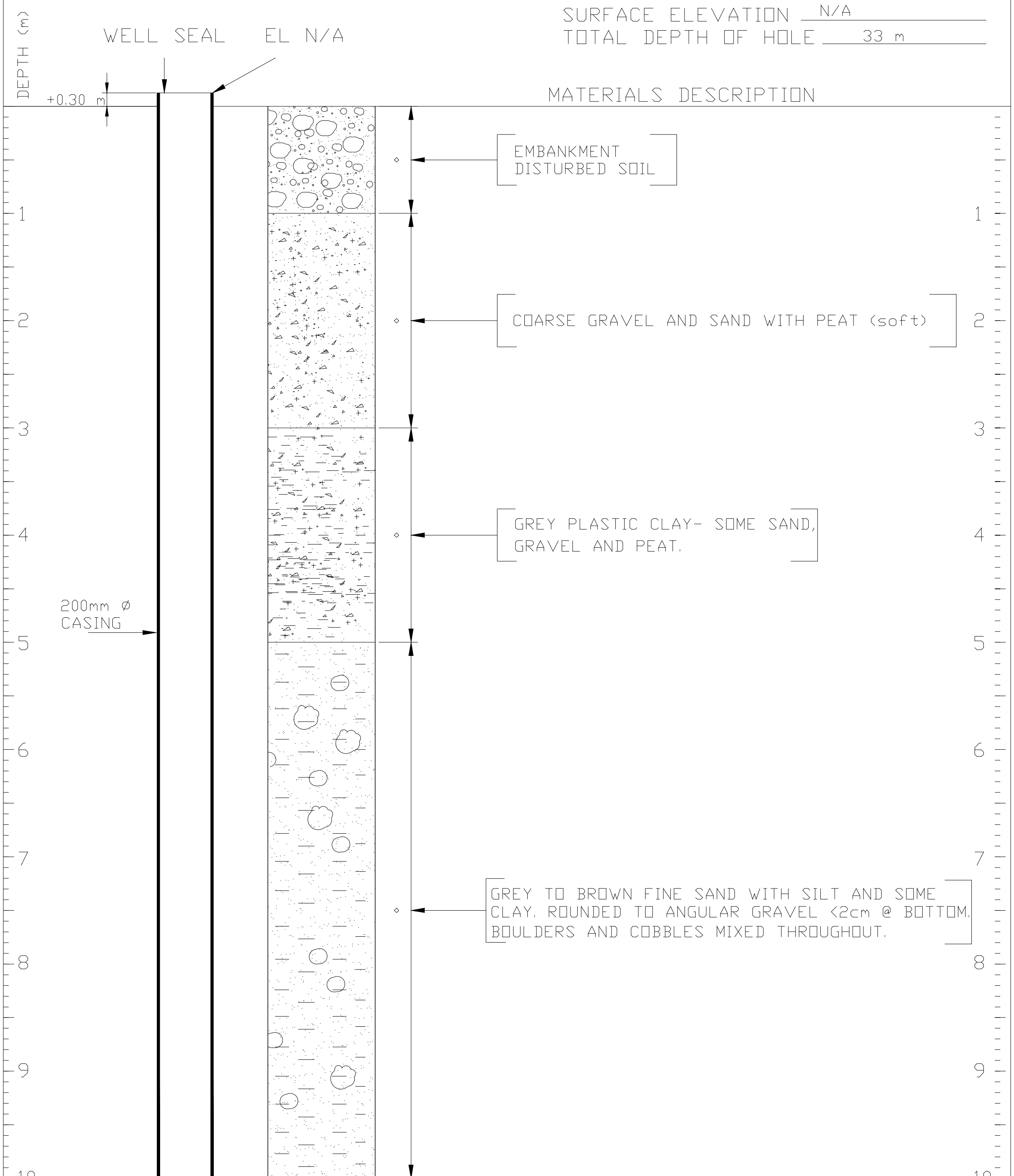
RED-BROWN AND GREY MUDSTONE. BEDROCK

31
32
33
34
35
26
37
38
39
40

FRACFLOW CONSULTANTS INC.

# BORING BH- PTW-4

CLIENT NAME STEPHENVILLE
DATE DRILLED NOV 9,10/98
SURFACE ELEVATION N/A
TOTAL DEPTH OF HOLE 33 m



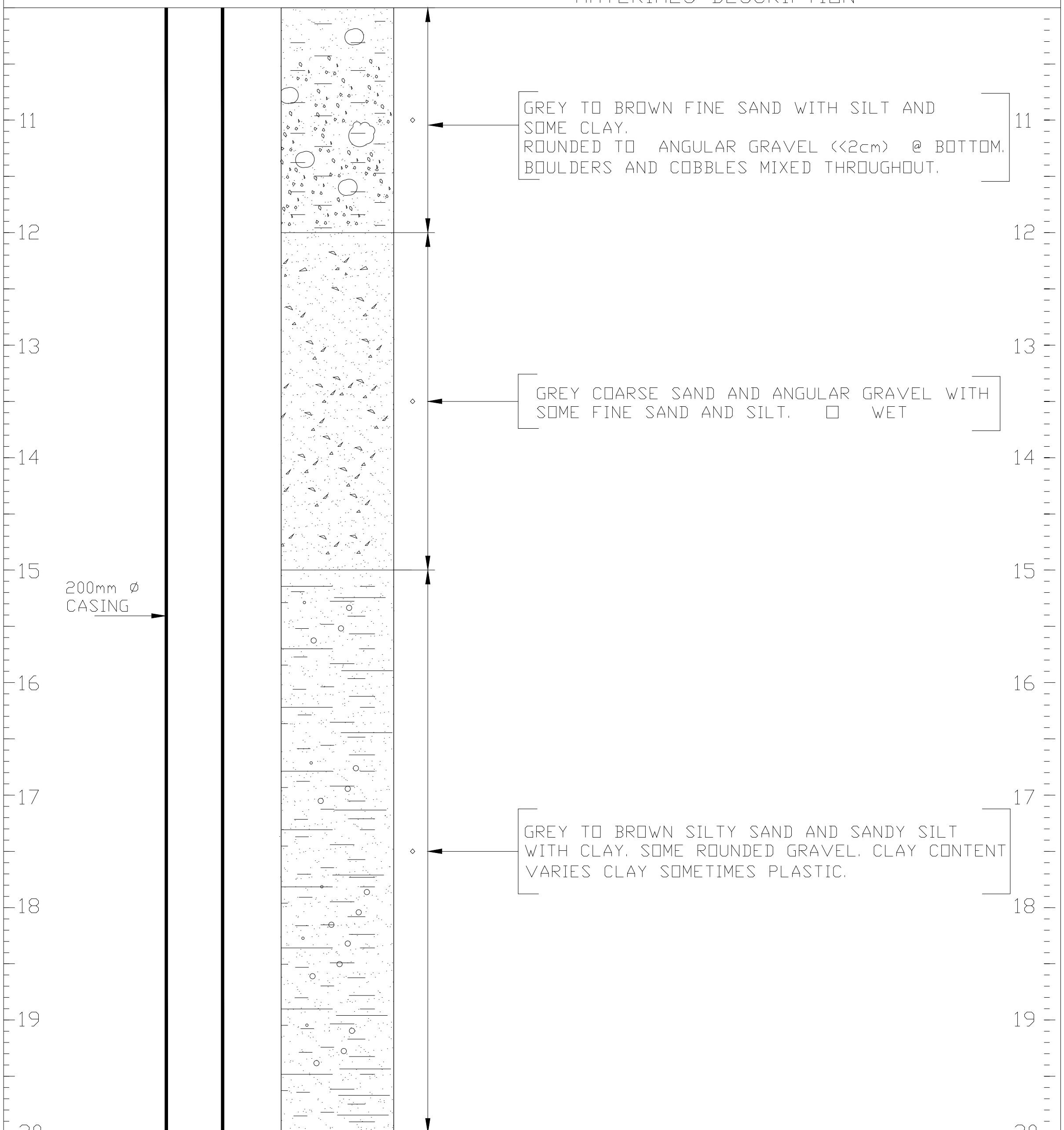

FRACFLOW CONSULTANTS INC.

# BORING BH- PTW-4

CLIENT NAME STEPHENVILLE
DATE DRILLED NOV 9,10/98
SURFACE ELEVATION N/A
TOTAL DEPTH OF HOLE 33 m

DEPTH (m)

MATERIALS DESCRIPTION




GREY TO BROWN FINE SAND WITH SILT AND SOME CLAY.
ROUNDED TO ANGULAR GRAVEL (<2cm) @ BOTTOM.
BOULDERS AND COBBLES MIXED THROUGHOUT.

GREY COARSE SAND AND ANGULAR GRAVEL WITH SOME FINE SAND AND SILT. □ WET

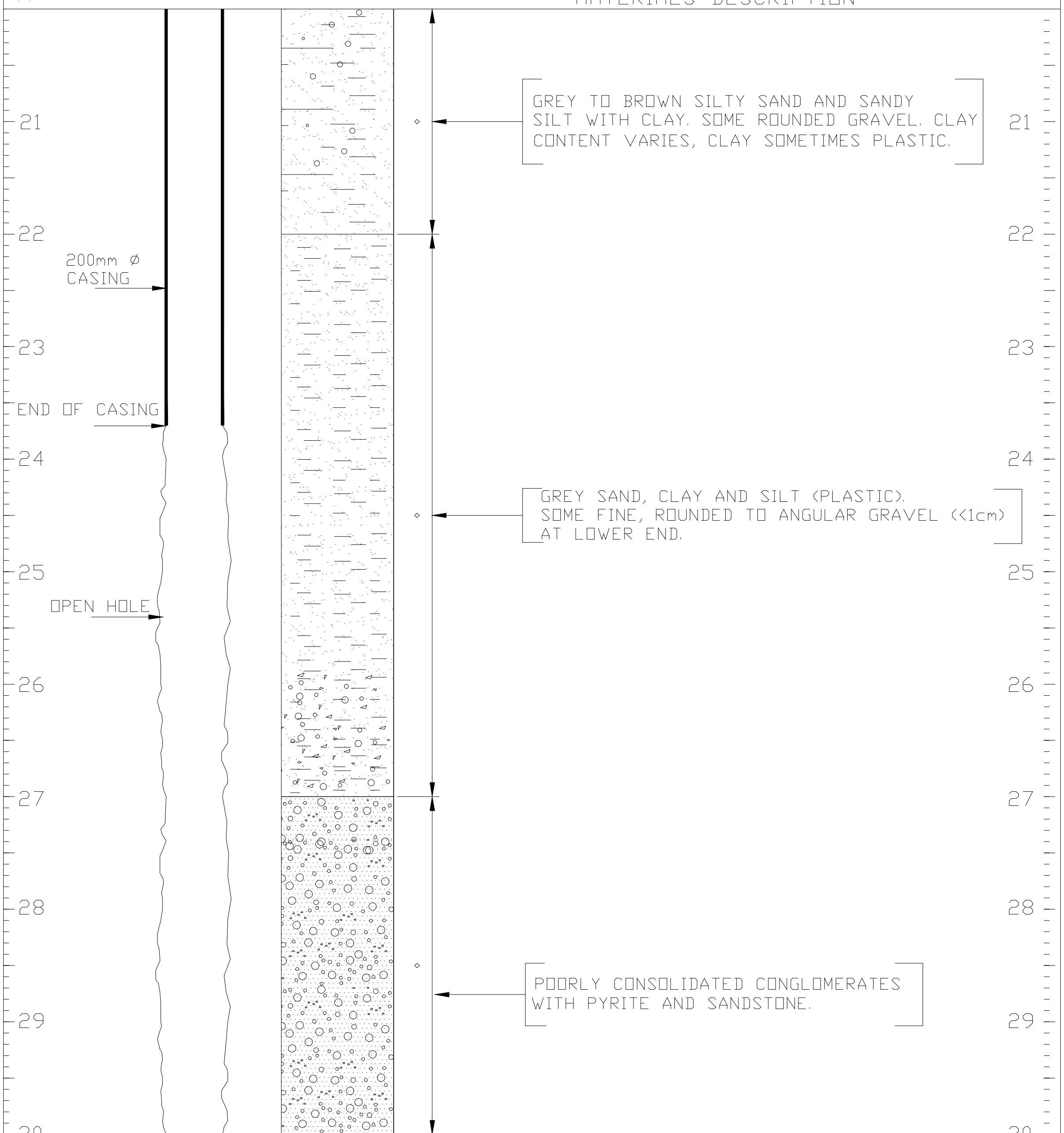
GREY TO BROWN SILTY SAND AND SANDY SILT WITH CLAY. SOME ROUNDED GRAVEL. CLAY CONTENT VARIES CLAY SOMETIMES PLASTIC.

# BORING BH- PTW-4

CLIENT NAME STEPHENVILLE
DATE DRILLED NOV 9,10/98
SURFACE ELEVATION N/A
TOTAL DEPTH OF HOLE 33 m

DEPTH (m)

MATERIALS DESCRIPTION



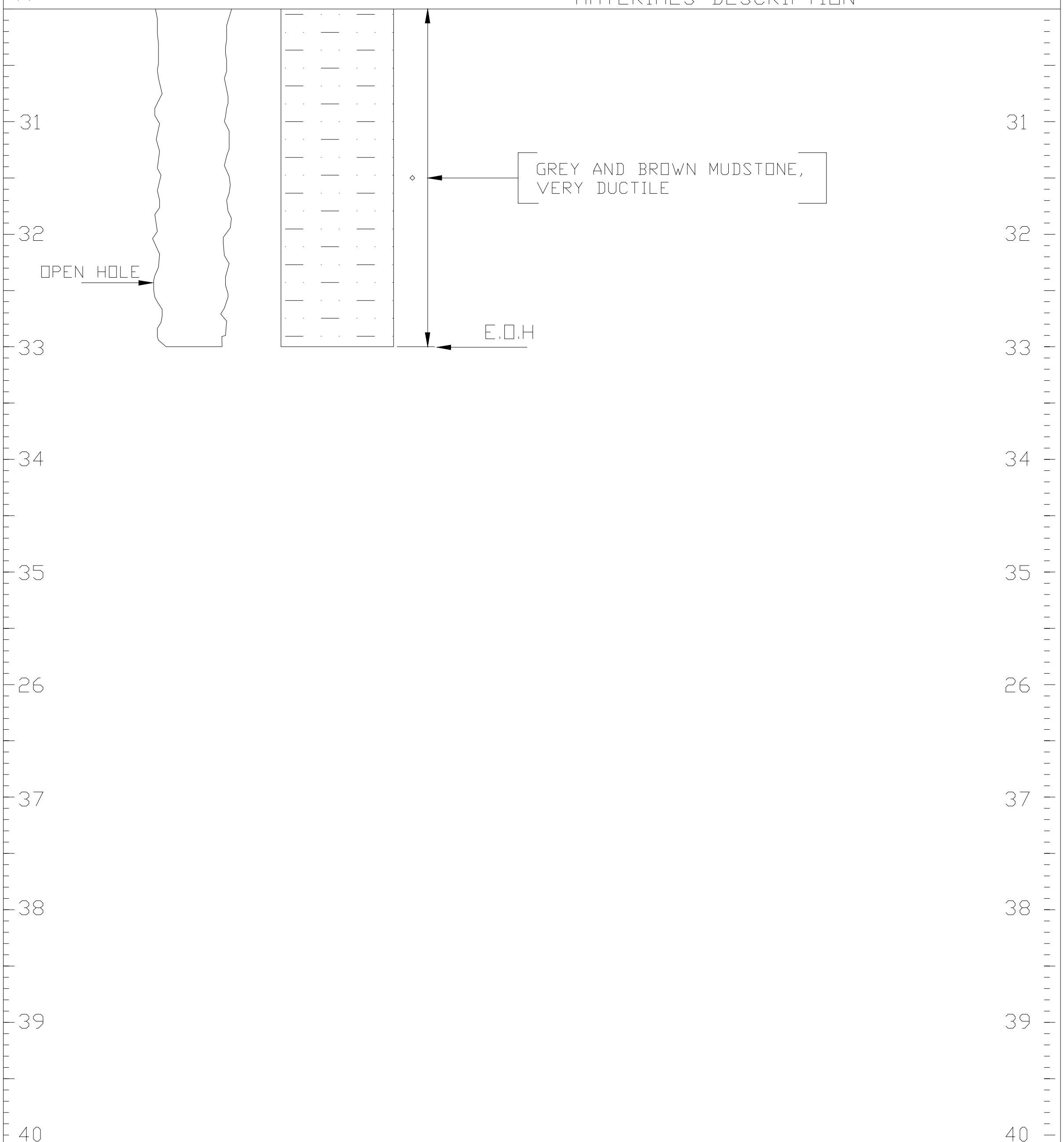

FRACFLOW CONSULTANTS INC.

# BORING BH- PTW-4

CLIENT NAME STEPHENVILLE
DATE DRILLED NOV 9,10/98
SURFACE ELEVATION N/A
TOTAL DEPTH OF HOLE 33 m

DEPTH (m)

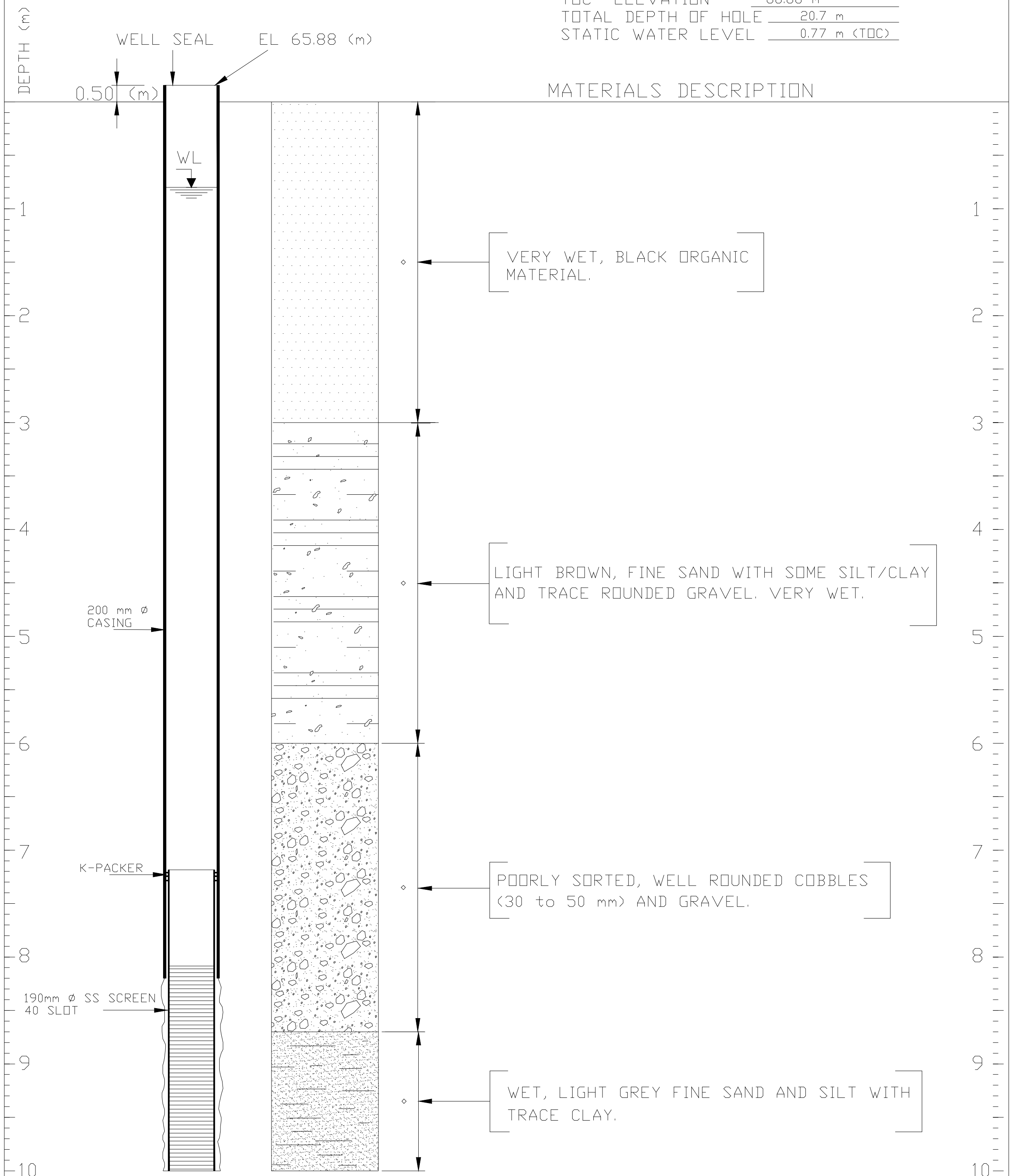
MATERIALS DESCRIPTION




FRACFLOW CONSULTANTS INC.

# BORING BH- PTW-5

CLIENT NAME STEPHENVILLE
DATE DRILLED DEC 2/98
TOC ELEVATION 68.88 m
TOTAL DEPTH OF HOLE 20.7 m
STATIC WATER LEVEL 0.77 m (TOC)



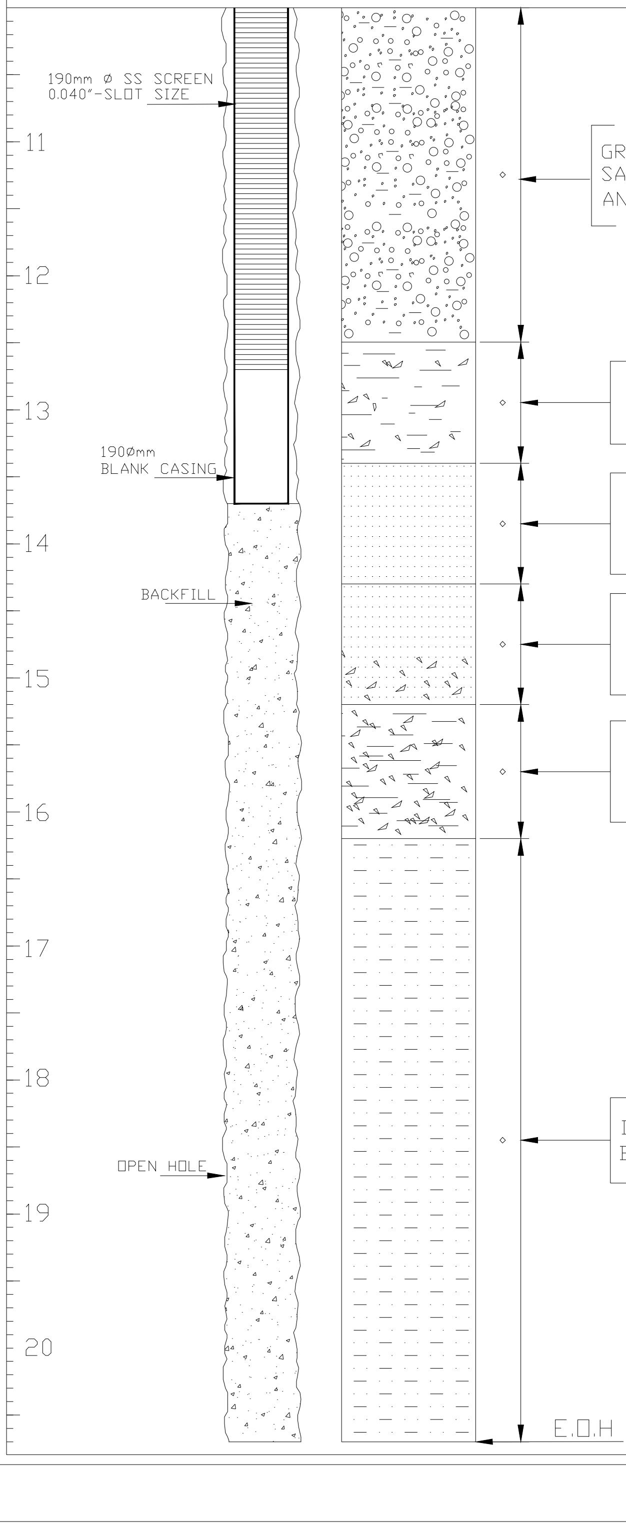

FRACFLOW CONSULTANTS INC.

# BORING BH- PTW-5

CLIENT NAME STEPHENVILLE
DATE DRILLED DEC 2/98
TOC ELEVATION 68.88 m
TOTAL DEPTH OF HOLE 20.7 m
STATIC WATER LEVEL 0.77 m (TOC)

DEPTH (m)

MATERIALS DESCRIPTION




GRAVEL AND COARSE ANGULAR TO SUB-ANGULAR SAND WITH WELL ROUNDED COBBLES (20 to 50 mm) AND LIGHT GREY TRACE SILT/CLAY.

WET, GRAY CLAY AND SILT WITH TRACE GRAVEL.

REDDISH-BROWN PLASTIC CLAY WITH TRACE SILT AND FINE SAND. DAMP.

REDDISH-BROWN, PLASTIC CLAY WITH TRACE SILT AND FINE GRAVEL. DAMP.

REDDISH MUD/SILTSTONE FRAGMENTS WITH REDDISH PLASTIC CLAYS. DAMP.

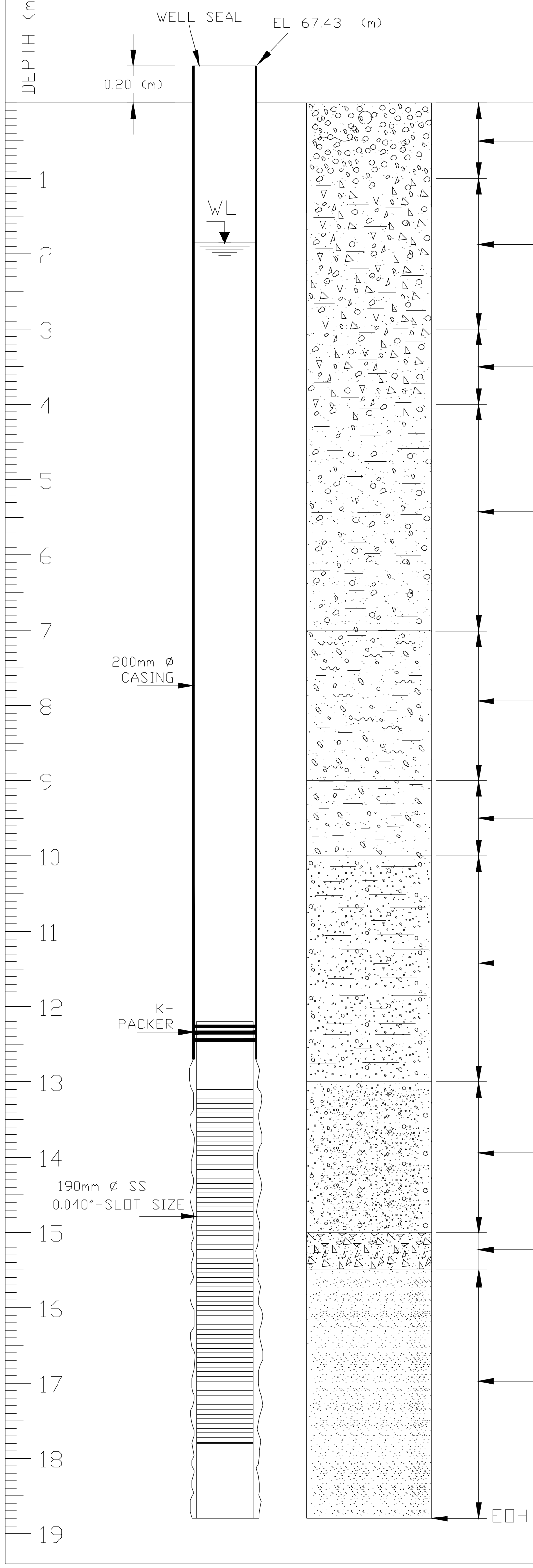
DRY, REDDISH MUDSTONE/SILTSTONE. BEDROCK

FRACFLOW CONSULTANTS INC.

PAGE (1 OF 1)

# BORING BH- PTW-6

CLIENT NAME: STEPHENVILLE
DATE DRILLED: JAN 13-14, 1999
TOC ELEVATION: 67.43 m
TOTAL DEPTH OF HOLE: 18.8 m
STATIC WATER LEVEL: 1.86 m (TOC)




MATERIALS DESCRIPTION

EMBANKMENT/BACKFILL

WET, DARK BROWN COARSE SAND AND CLAY WITH ANGULAR TO ROUND GRAVELS, COBBLES AND BOULDERS.

COARSE SAND WITH SOME ANGULAR GRAVEL, AND SOME FINE SAND.

BROWN-GREY COARSE SAND WITH SUB-ANGULAR GRAVEL. SOME FINE SAND, SILT AND CLAY. (DAMP)

BROWN, ROUND GRAVEL AND FINE SAND. SOME CLAY AND SILT. (PLASTIC)

CLAYEY SILT WITH SOME FINE SAND AND ROUNDED GRAVEL.

FINE TO COARSE SAND WITH POORLY ROUNDED GRAVEL (2 to 7 cm). SOME SILT AND CLAY. (DAMP)

BROWN-GREY COARSE SAND WITH FINE SILTY SAND AND TRACE OF FINE (1cm) ROUNDED GRAVEL. (DAMP)

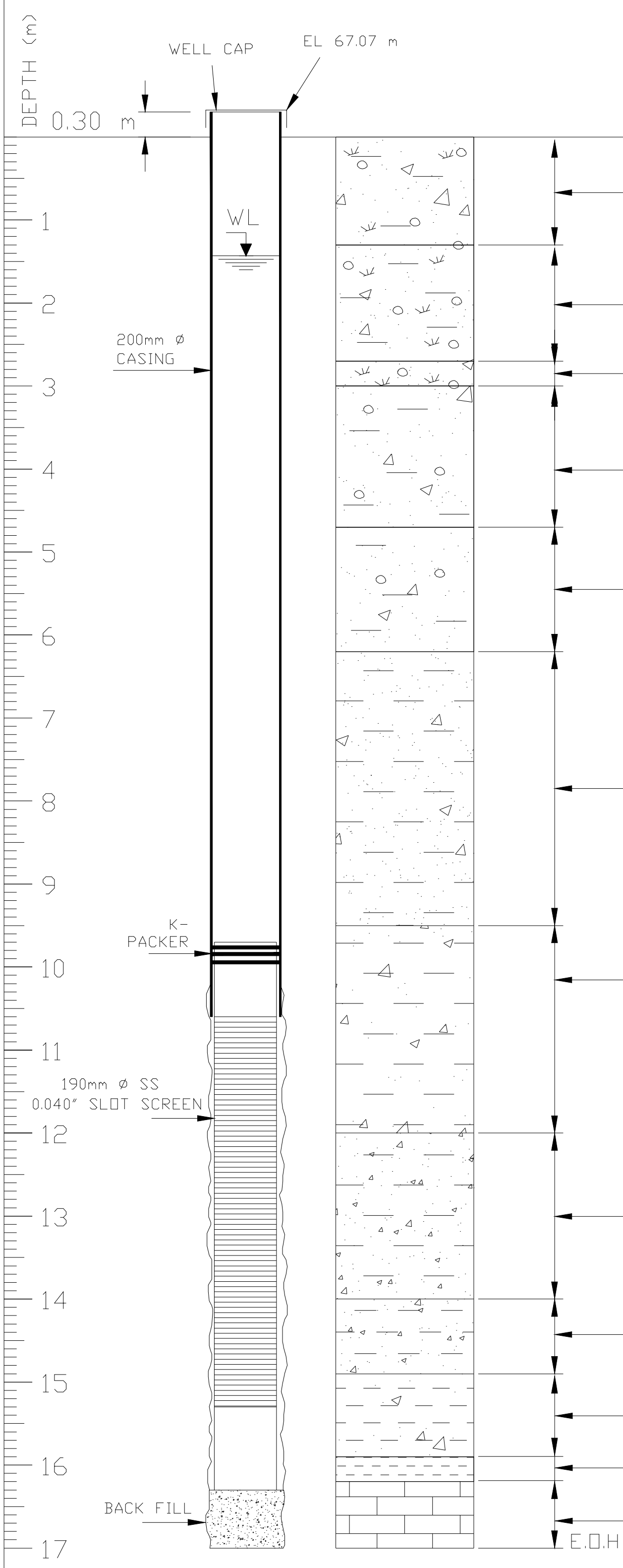
COMPACT, COARSE SAND AND GRAVEL.

HARD, COARSE GRAINED GREY SANDSTONE.

FRACFLOW CONSULTANTS INC.

# BORING BH- PTW-7

CLIENT NAME STEPHENVILLE
DATE DRILLED JAN 17/1999
TOC ELEVATION 67.07 m
TOTAL DEPTH OF HOLE 17 m
STATIC WATER LEVEL 1.43 m (TOC)




## MATERIALS DESCRIPTION

DARK BROWN SAND, FINES, ORGANICS AND ROUNDED GRAVEL/COBBLES.

VERY SOFT, DARK BROWN TO BLACK WATERY ORGANICS (PEAT) WITH TRACE SAND.

PEAT MIXED WITH ROUNDED GRAVEL AND SAND.

GREY-BROWN ROUNDED GRAVEL WITH COARSE SAND, SOME FINES.

LIGHT BROWN ANGULAR GRAVEL AND COARSE TO FINE SAND, TRACE FINES.

VERY SOFT CLAY, SILT AND SAND. TRACE GRAVEL.

GREY-BROWN ANGULAR AND ROUNDED GRAVEL AND COARSE TO FINE SAND. SOME FINES.

DARK GREY COARSE SAND WITH ROUNDED TO ANGULAR GRAVEL. SOME FINE SAND WITH TRACE FINES.

DARK GREY COARSE TO FINE SILTY SAND WITH ROUNDED GRAVEL. SOME CLAY.

RED TO BROWN SILT WITH TRACES OF FINE GRAVEL.

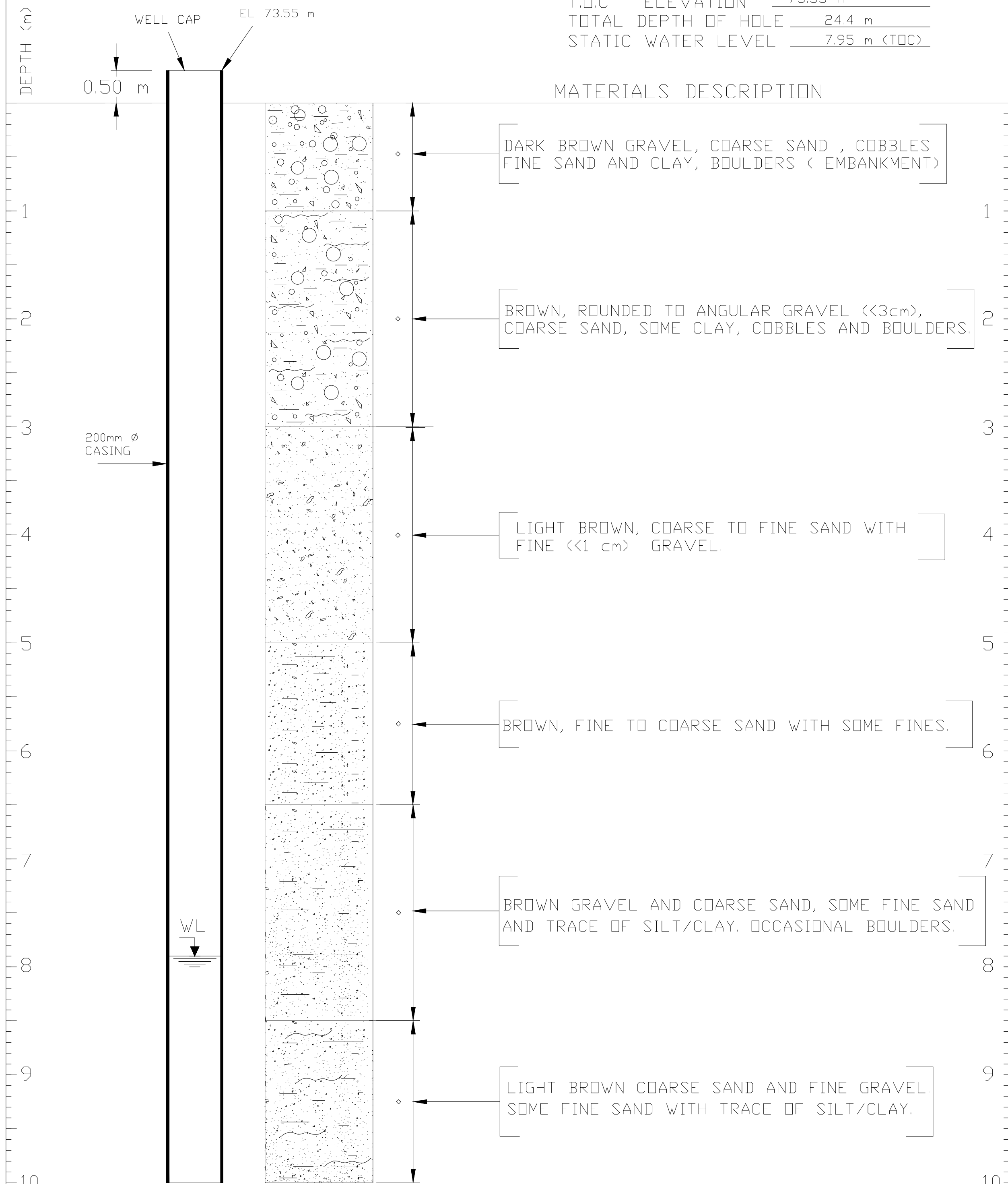
LIGHT GREY CLAY ( PLASTIC )

LIGHT BROWN TO RED-BROWN MUDSTONE. ( BEDROCK )

FRACFLOW CONSULTANTS INC.

# BORING BH- PTW-8

CLIENT NAME STEPHENVILLE
DATE DRILLED FEB 10-11 /99
T.O.C ELEVATION 73.55 m
TOTAL DEPTH OF HOLE 24.4 m
STATIC WATER LEVEL 7.95 m (TOC)




MATERIALS DESCRIPTION

DARK BROWN GRAVEL, COARSE SAND , COBBLES FINE SAND AND CLAY, BOULDERS ( EMBANKMENT)

BROWN, ROUNDED TO ANGULAR GRAVEL (<3cm), COARSE SAND, SOME CLAY, COBBLES AND BOULDERS.

LIGHT BROWN, COARSE TO FINE SAND WITH FINE (<1 cm) GRAVEL.

BROWN, FINE TO COARSE SAND WITH SOME FINES.

BROWN GRAVEL AND COARSE SAND, SOME FINE SAND AND TRACE OF SILT/CLAY. OCCASIONAL BOULDERS.

LIGHT BROWN COARSE SAND AND FINE GRAVEL. SOME FINE SAND WITH TRACE OF SILT/CLAY.

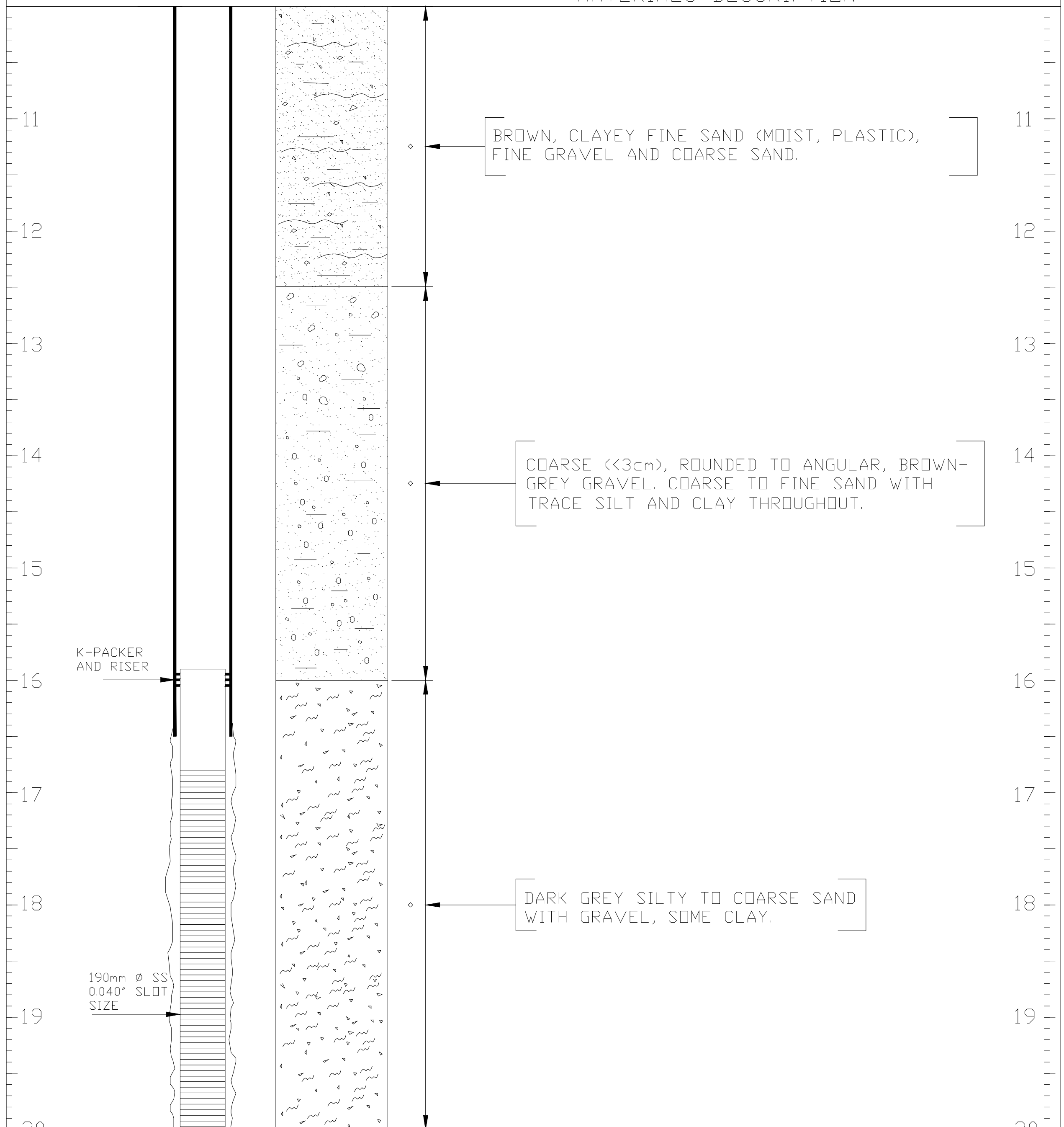
FRACFLOW CONSULTANTS INC.

# BORING BH- PTW-8

CLIENT NAME STEPHENVILLE
DATE DRILLED FEB 10-11 /99
T.O.C ELEVATION 73.55 m
TOTAL DEPTH OF HOLE 24.4 m
STATIC WATER LEVEL 7.95 m (TOC)

DEPTH (m)

MATERIALS DESCRIPTION

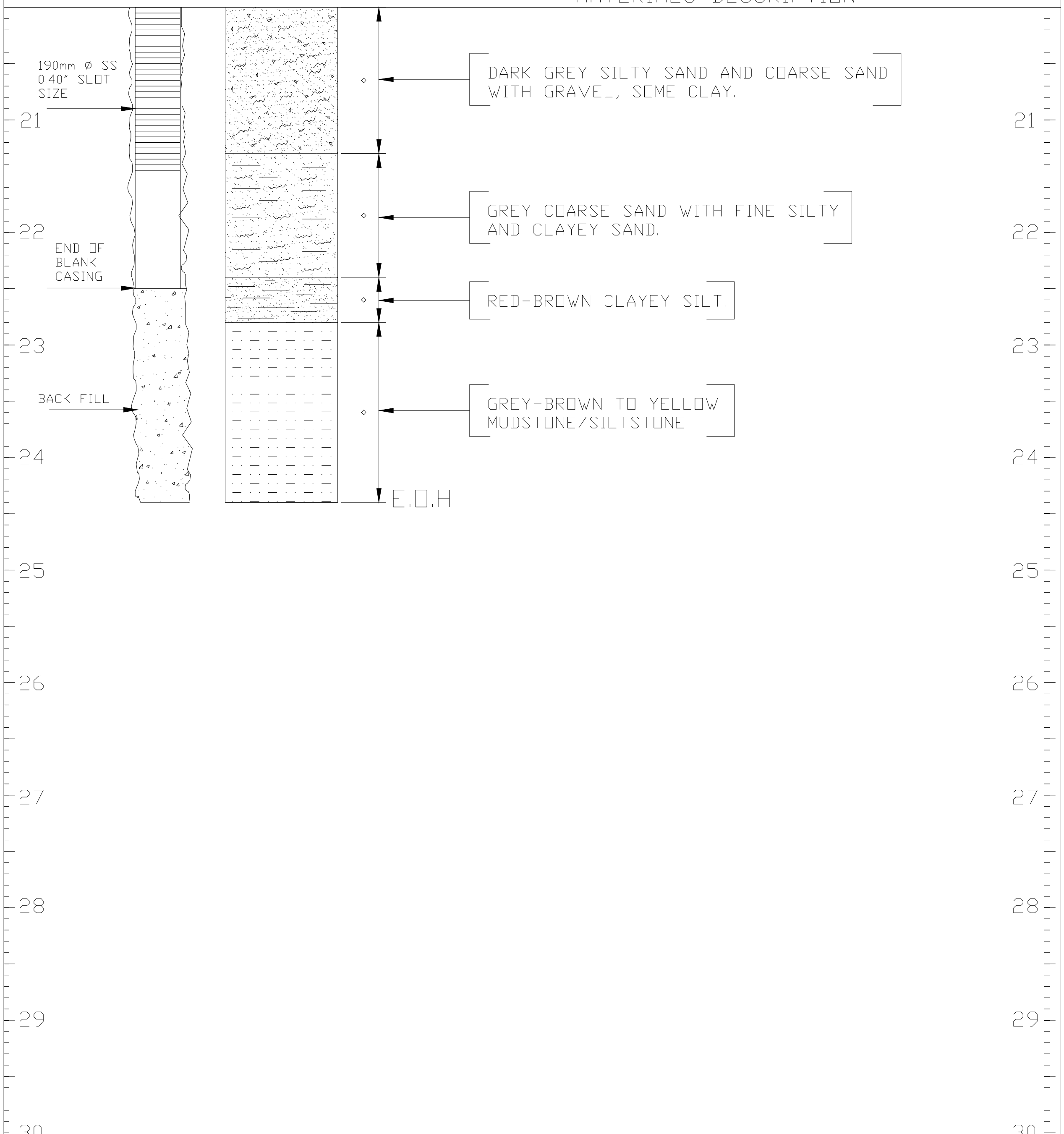


# BORING BH- PTW-8

CLIENT NAME STEPHENVILLE
DATE DRILLED FEB 10-11 /99
T.O.C ELEVATION 73.55 m
TOTAL DEPTH OF HOLE 24.4 m
STATIC WATER LEVEL 7.95 m (TOC)

DEPTH (m)

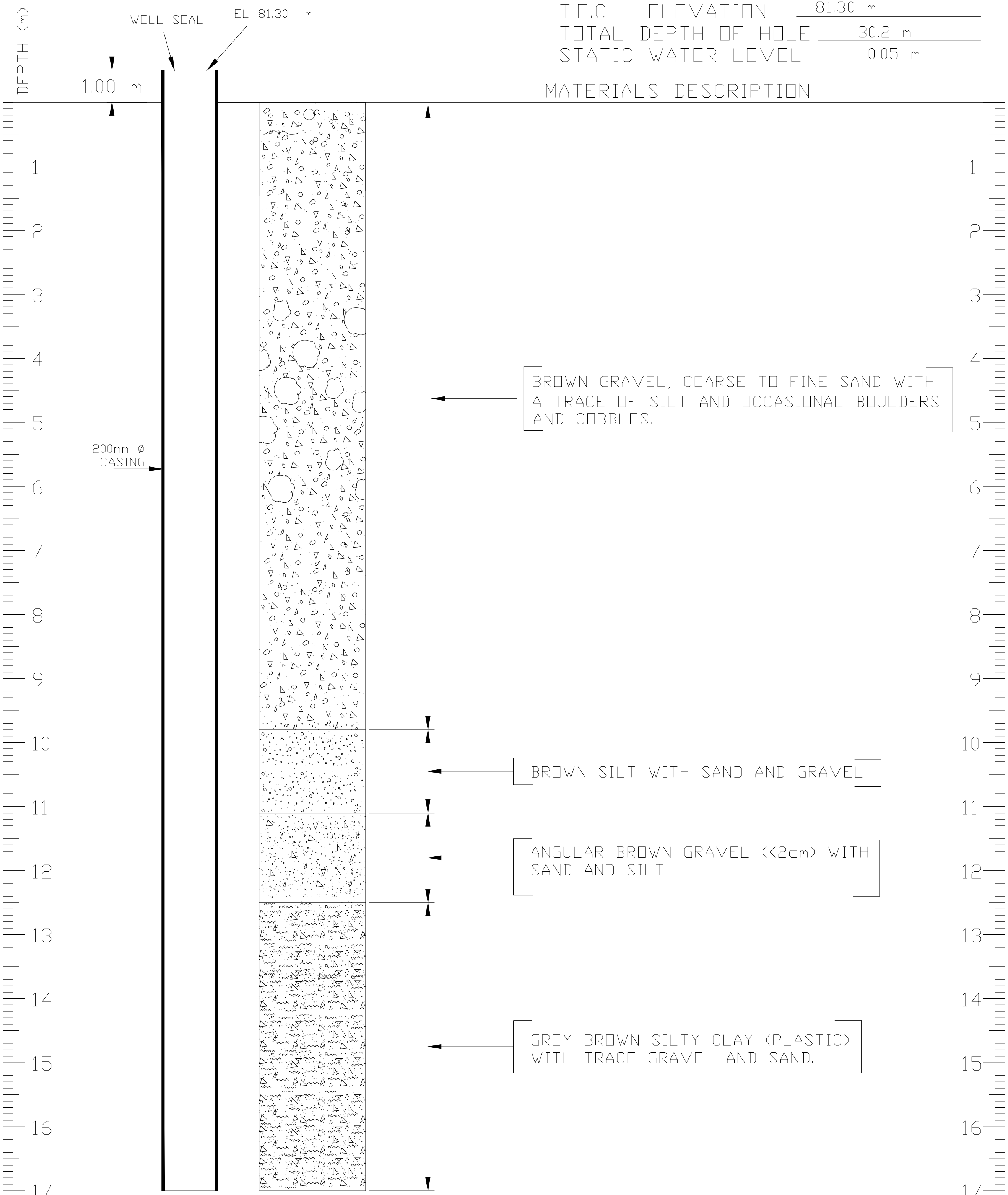
MATERIALS DESCRIPTION




FRACFLOW CONSULTANTS INC.

# BORING BH- PTW-9

CLIENT NAME STEPHENVILLE
DATE DRILLED FEB 16-17/1999
T.O.C ELEVATION 81.30 m
TOTAL DEPTH OF HOLE 30.2 m
STATIC WATER LEVEL 0.05 m

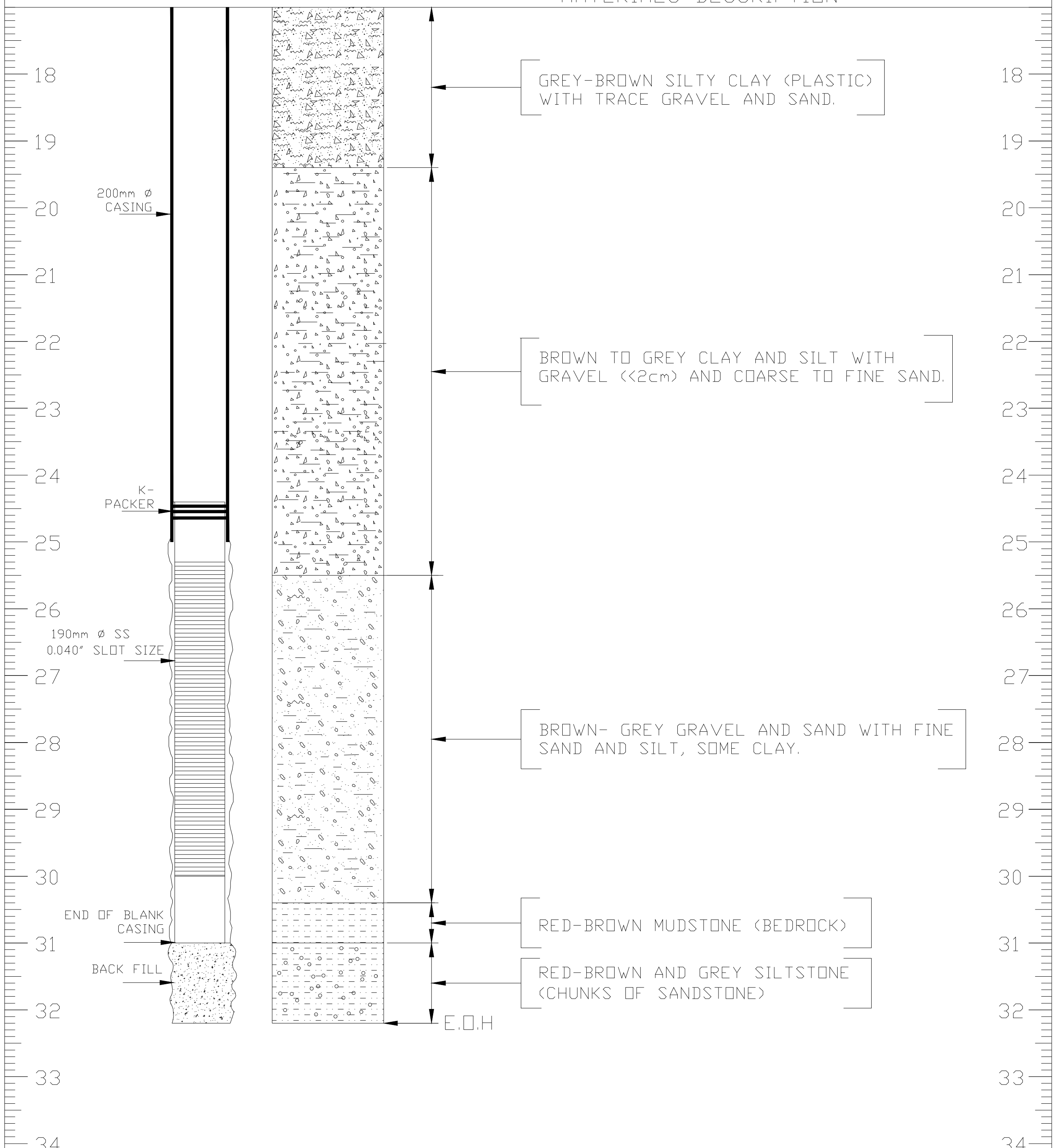


# BORING BH- PTW-9

CLIENT NAME: STEPHENVILLE
DATE DRILLED: FEB 16-17/1999
TOC ELEVATION: 81.30 m
TOTAL DEPTH OF HOLE: 30.2 m
STATIC WATER LEVEL: 0.05 m

DEPTH (m)

MATERIALS DESCRIPTION

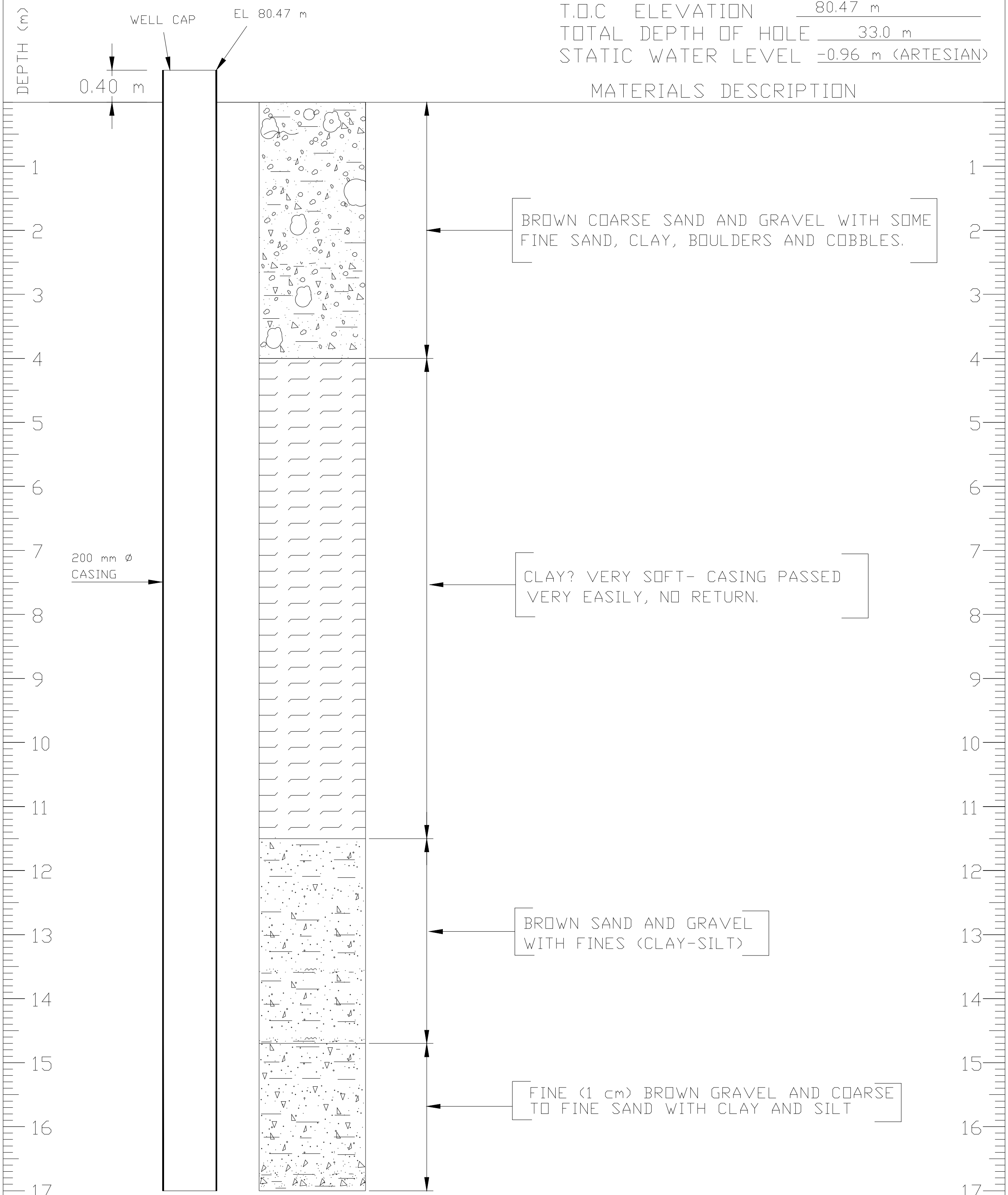



FRACFLOW CONSULTANTS INC.

PAGE (1 OF 2)

# BORING BH- PTW-10

CLIENT NAME STEPHENVILLE
DATE DRILLED FEB 18-19/1999
T.O.C ELEVATION 80.47 m
TOTAL DEPTH OF HOLE 33.0 m
STATIC WATER LEVEL -0.96 m (ARTESIAN)



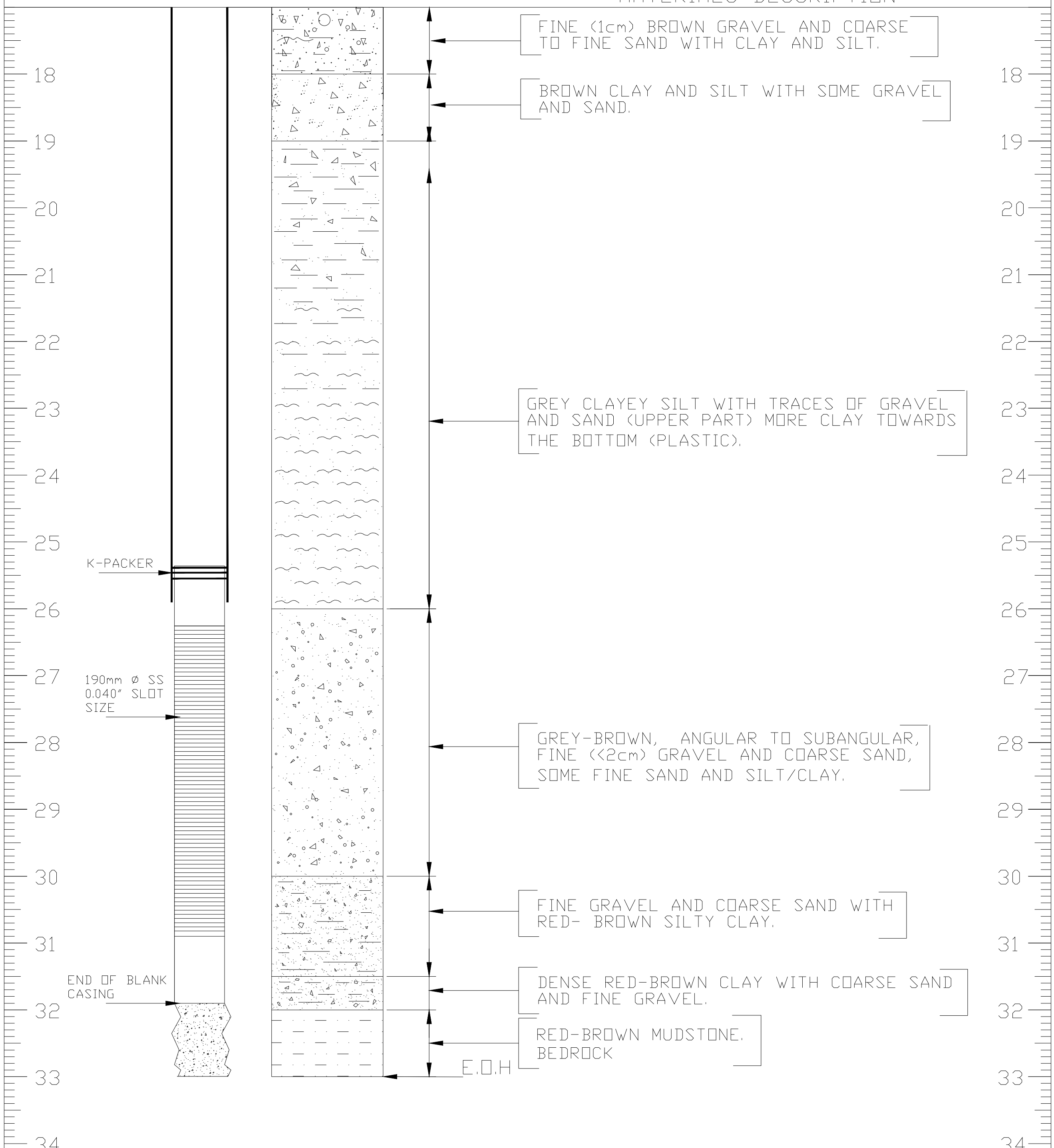

FRACFLOW CONSULTANTS INC.

# BORING BH- PTW-10

CLIENT NAME STEPHENVILLE
DATE DRILLED FEB 18-19/1999
SURFACE ELEVATION N/A
TOTAL DEPTH OF HOLE 33.0 m

DEPTH (m)

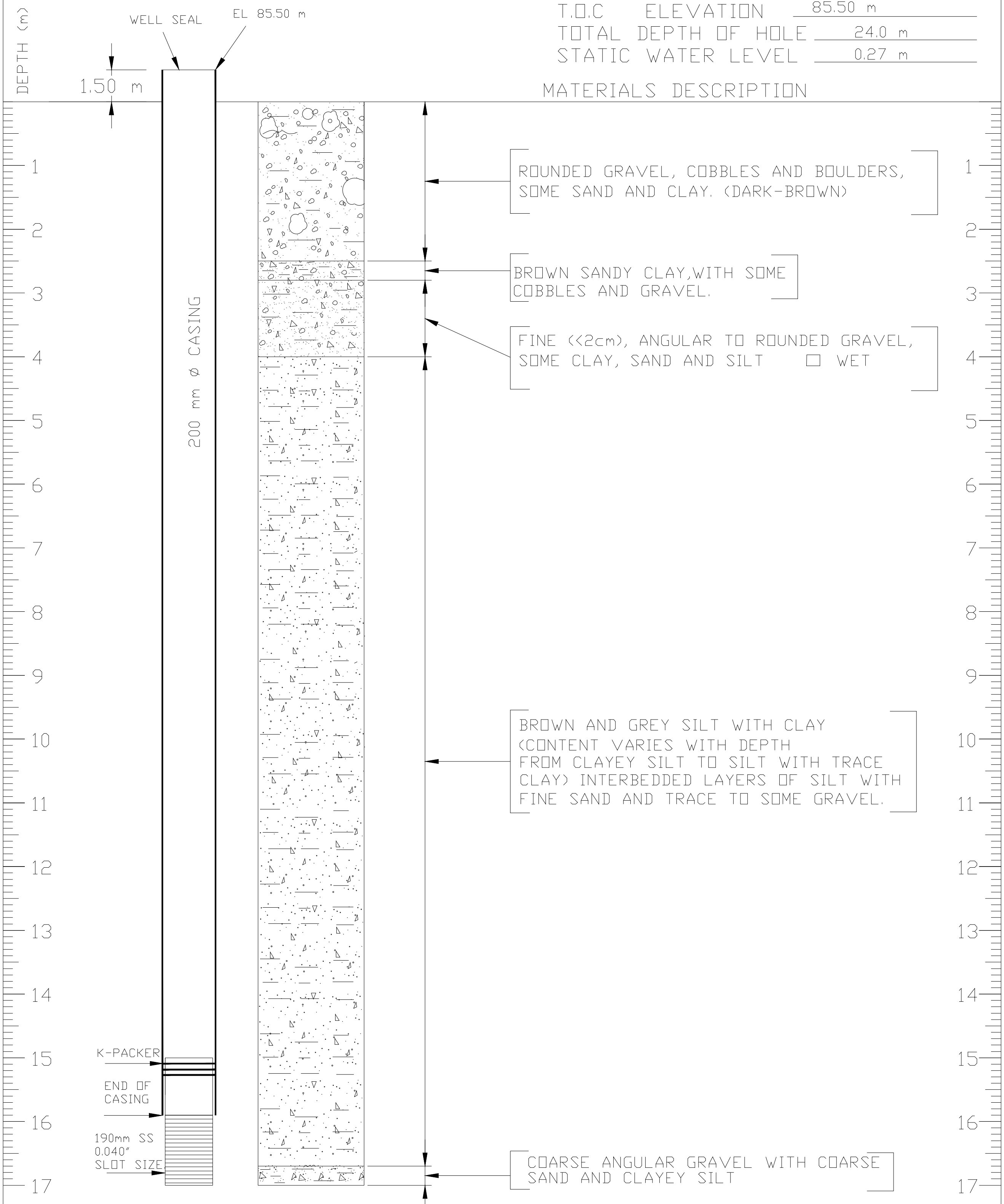
MATERIALS DESCRIPTION




FRACFLOW CONSULTANTS INC.

# BORING BH- PTW-11

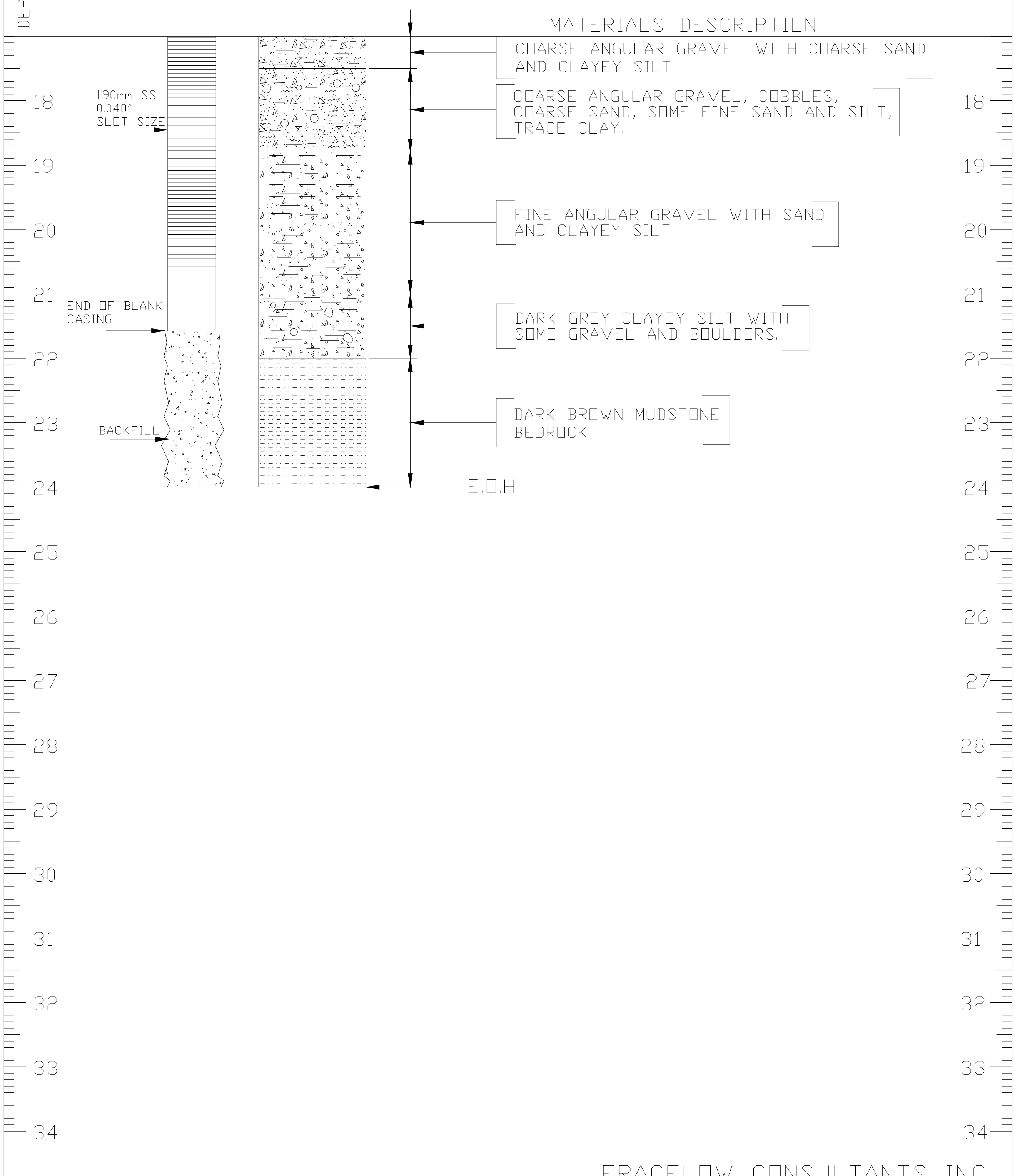
CLIENT NAME STEPHENVILLE
DATE DRILLED MARCH 18/1999
T.O.C ELEVATION 85.50 m
TOTAL DEPTH OF HOLE 24.0 m
STATIC WATER LEVEL 0.27 m




FRACFLOW CONSULTANTS INC.

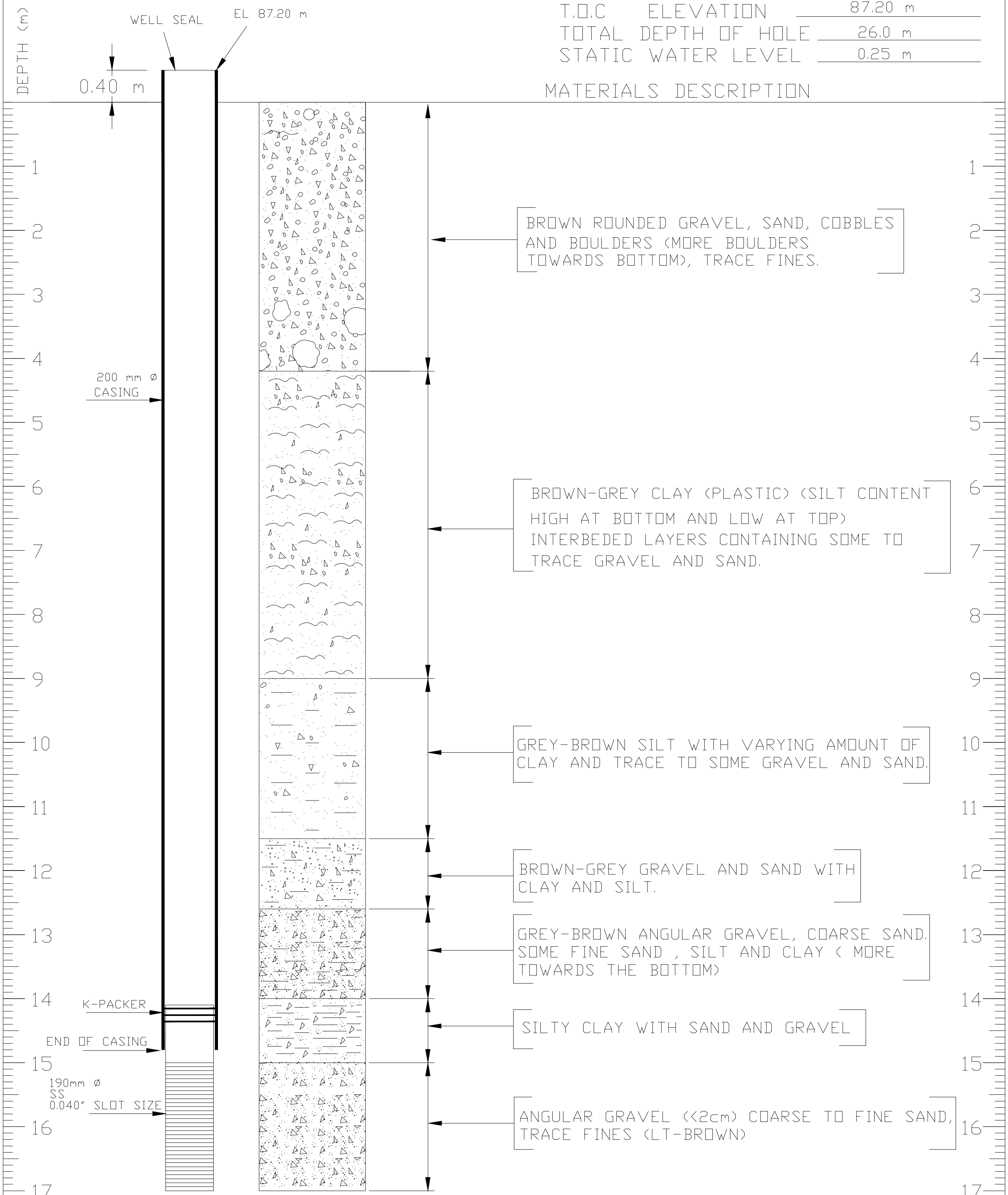
# BORING BH- PTW-11

CLIENT NAME STEPHENVILLE
DATE DRILLED MARCH 18/1999
T.O.C ELEVATION 85.50 m
TOTAL DEPTH OF HOLE 24.0 m



# BORING BH- PTW-12

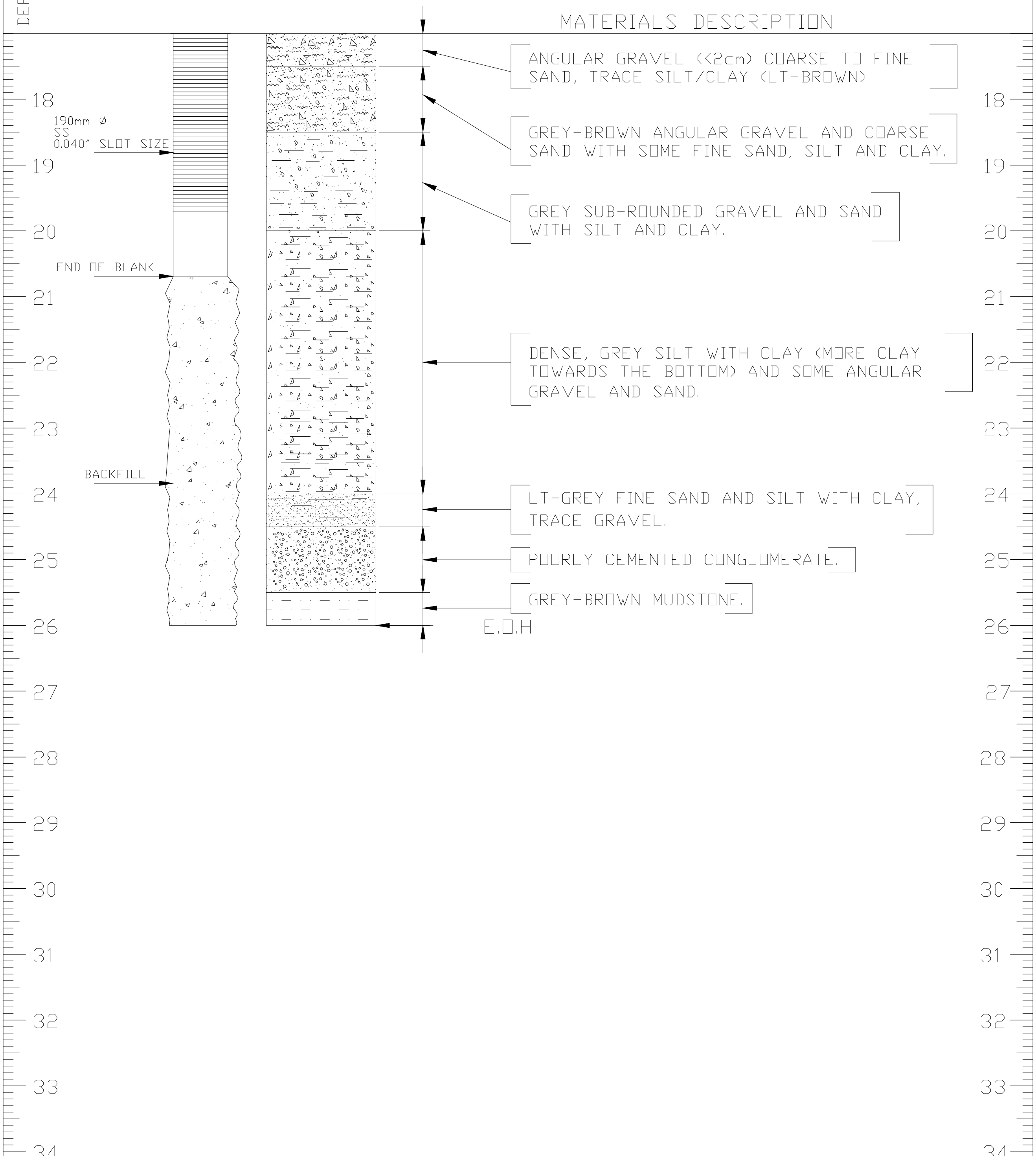
CLIENT NAME STEPHENVILLE
DATE DRILLED MARCH 24-27/1999
T.O.C ELEVATION 87.20 m
TOTAL DEPTH OF HOLE 26.0 m
STATIC WATER LEVEL 0.25 m




FRACFLOW CONSULTANTS INC.

# BORING BH- PTW-12

CLIENT NAME: STEPHENVILLE
DATE DRILLED: MARCH 24-27/1999
TOC ELEVATION: 87.20 m
TOTAL DEPTH OF HOLE: 26.0 m
STATIC WATER LEVEL: 0.25 m



Project: Water Suppy Well

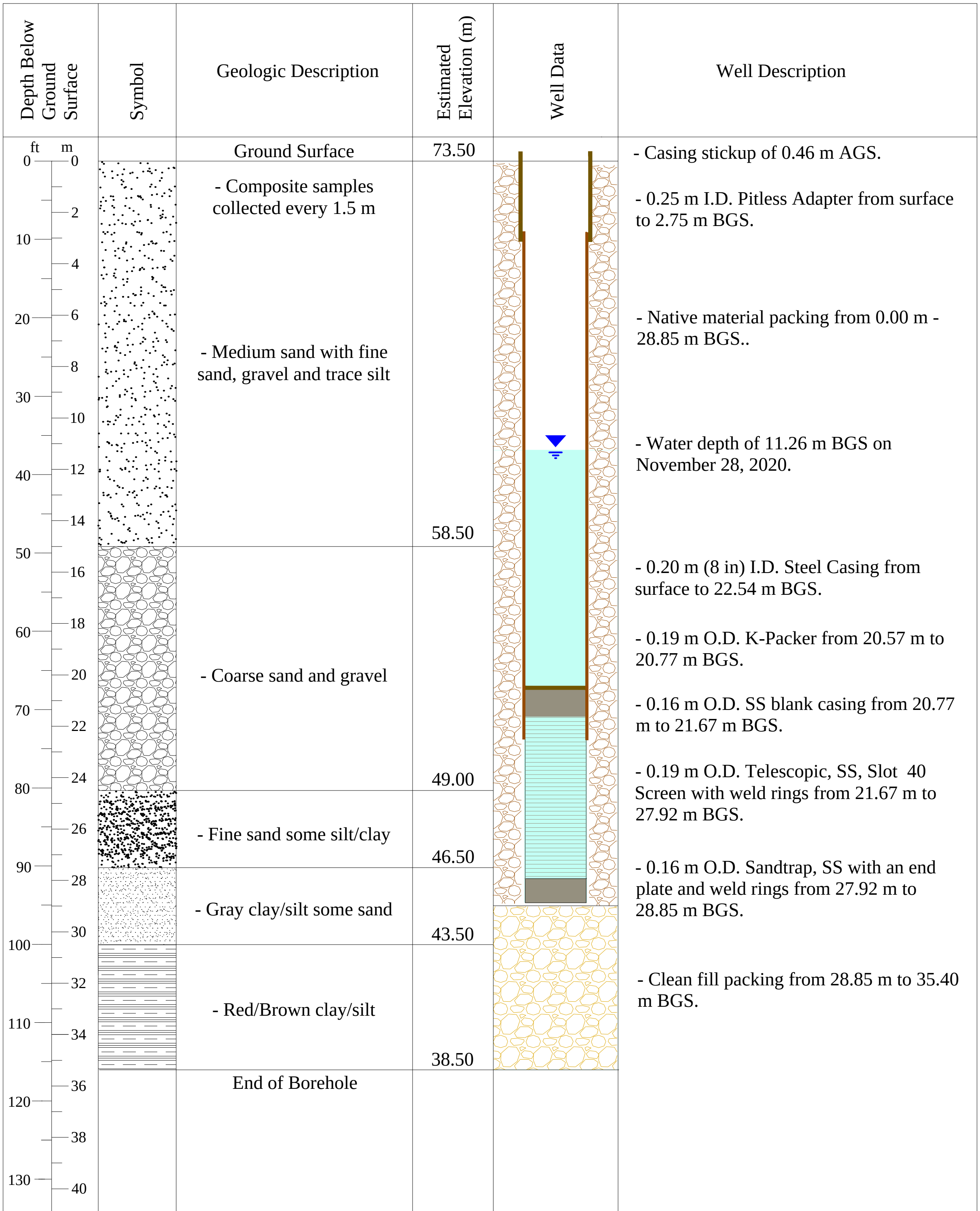
Location: Stephenville, NL

Client: Town of Stephenville

## *Well Log: Well5*

Project No: 555

Date: November 14 - 15, 2020




Borehole and well construction log for new Well5 (Source: FFC-NL-555-003).

| Project No. | Document No. |
|---|---|
| 555 | FFC-NL-555-004 |
| **Location** | **Date** |
| Stephenville | July 2021 |

FFC