Modeling and Designing a Hydraulic Source Zone Isolation System

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MODELING AND DESIGNING A HYDRAULIC SOURCE ZONE ISOLATION SYSTEM

A Thesis
Presented to
the Graduate School of
Clemson University

In Partial Fulfillment
of the Requirements for the Degree
Master of Science
Hydrogeology

by
Yoel Gebrai
August 2018

Accepted by:
Dr. Ronald Falta, Committee Chair
Dr. David Freedman
Dr. Lawrence Murdoch
Abstract

Pump and Treat technology remains one of the most common approaches for groundwater remediation at contaminated sites. Despite its prevalence, the effectiveness of the Pump and Treat approach is limited by the chemical properties of the contaminant, heterogeneity, and cost (Mackay and Cherry, 1989; NRC, 1994). Pump and Treat systems are often in operation for long periods of time, at which point their environmental footprint, in addition to cost, needs to be considered. Greenhouse gas emissions from electricity generation to power pumps and the disruption of ecosystems over time are undesirable environmental effects that can be a cause of concern at sites that employ Pump and Treat (Ellis and Hadley, 2009; USEPA, 2009).

A potentially more sustainable alternative to Pump and Treat remediation is source zone isolation. Source zone isolation can be achieved by surrounding a contaminant source with a material of contrasting permeability. The construction of an impermeable barrier, such as a sheet pile or slurry wall, is one technique that can be used to contain a contaminant source. However impermeable barriers must typically be excavated to a confining layer or bedrock and require nearly perfect construction to be effective. Impermeable barriers may also create a scenario where groundwater mounding takes place.

A permeable hydraulic barrier, such as a French drain (Davis and Stansfield, 1984) or constant head trench (Ankeny and Forbes, 1997), can provide a cheaper and
more robust means for source zone isolation than an impermeable barrier. Clean, up-
stream groundwater would preferentially flow into, through, and exit the permeable
barrier without contacting the source zone and contributing to an existing plume.
This study utilizes groundwater flow and contaminant transport modeling to assess
the influence of various permeable hydraulic barrier design parameters such as thick-
ness, effective hydraulic conductivity, depth, and the inclusion of wells on source zone
isolation. Additionally, different hydrogeologic scenarios for a prospective site are
explored to see how they impact the performance of a permeable hydraulic barrier.

Sensitivity analyses were performed on different design parameters in the pres-
ence of preexisting plumes generated from a continuous source in a homogeneous
aquifer conceptual model and in a heterogeneous one as well. The simulation re-
results for a homogeneous aquifer conceptual model suggests that permeable hydraulic
barrier designs should fully penetrate an aquifer in order to achieve optimal results.
Increases in the effective hydraulic conductivity and trench thickness are shown to
increase hydraulic isolation. Simulation results of a sensitivity analysis performed on
a heterogeneous aquifer conceptual model indicate that the presence of heterogeneity
may enhance the performance of a hydraulic barrier.

Designs such as a gravel trench, a gravel trench with pipe, and a gravel trench
with pipe and source zone pumping were simulated for aquifer models of a prospective
site where different hydrogeologic scenarios are considered. Calibrated groundwater
models were constructed for scenarios such as an aquifer overlying intact bedrock,
an aquifer intersected by a highly conductive layer and overlying intact bedrock, and
an aquifer overlying fractured bedrock. A highly heterogeneous aquifer model was
also constructed and considered for a case where the bedrock is intact and where
the underlying bedrock is fractured as well. Each design considered reduced plume
mass by at least 79% for each hydrogeologic scenario. The inclusion of pipe in the
gravel trench outperformed the gravel trench design by 1% to 7%. Aquifer scenarios that have intact bedrock were generally more responsive to the inclusion of pipe in the hydraulic barrier. For scenarios that considered an aquifer overlying fractured bedrock, the difference between the performance of a gravel trench with pipe and a gravel trench designs was less and source zone pumping was needed to improve the performance of the permeable hydraulic barrier designs.
Acknowledgments

I would like to thank my advisor, Dr. Ron Falta, for taking me on as a student to work with him and his guidance throughout this project. I have learned a lot in the process and my gratitude for this opportunity knows no bounds. I would also like to thank Dr. David Freedman and Dr. Larry Murdoch for their willingness to serve as my committee members and for providing valuable feedback on this work. This project would not have been possible without the support of Dr. James Henderson and the funding provided by DuPont.
# Table of Contents

Title Page ................................................................. i

Abstract ................................................................. ii

Acknowledgments ......................................................... v

List of Tables ............................................................ ix

List of Figures ........................................................... x

1 Introduction ........................................................... 1
   1.1 Emergence of Groundwater Contamination ....................... 1
   1.2 Groundwater Remediation Challenges ............................ 2
   1.3 Sustainable Remediation .......................................... 4
   1.4 Hydraulic Isolation ................................................ 5
   1.5 Prior Work .......................................................... 7
   1.6 Research Goals ..................................................... 14

2 Hydraulic Isolation System Design ................................. 16
   2.1 Gravel Trench ....................................................... 16
   2.2 Gravel Trench and Pipe ........................................... 17
   2.3 Gravel Trench, Pipe, and Pumping and Injection Wells in the Trench 21
   2.4 Gravel Trench, Pipe, and Source Zone Pumping .................. 22

3 Homogeneous Conceptual Model ..................................... 23
   3.1 Flow Conceptual Model ............................................ 23
   3.2 Groundwater Model ................................................ 25
   3.3 Hydraulic Barrier Inclusion ....................................... 26
   3.4 Hydraulic Barrier Simulation ..................................... 27
   3.5 Transport Conceptual Model ...................................... 28
   3.6 Sensitivity Analysis ............................................... 31
   3.7 Hydraulic Isolation Designs ..................................... 45

4 Heterogeneous Conceptual Model ................................. 55
4.1 TPROGS ............................................. 55
4.2 Groundwater Model ................................. 59
4.3 Hydraulic Barrier Inclusion ....................... 61
4.4 Contaminant Transport Model .................... 62
4.5 Sensitivity Analysis ............................... 65

5 Barra Mansa Site ................................. 78
  5.1 Location .......................................... 79
  5.2 Boundary Conditions ............................ 80
  5.3 Hydrogeology .................................... 83
  5.4 Recharge .......................................... 84
  5.5 Calibration Targets ............................. 86
  5.6 Discretization ................................... 88
  5.7 Transport Conceptual Model .................... 90
  5.8 Hydraulic Isolation Designs .................... 92

6 Intact Bedrock Scenario .......................... 96
  6.1 Groundwater Flow Calibration ................... 99
  6.2 Contaminant Transport ........................ 102
  6.3 Gravel Trench Results .......................... 105
  6.4 Gravel Trench with Pipes Results ............. 108
  6.5 Comparison ...................................... 110

7 Highly Conductive Bed Scenario ................. 112
  7.1 Groundwater Flow Calibration ................... 115
  7.2 Contaminant Transport ........................ 118
  7.3 Gravel Trench Results .......................... 120
  7.4 Gravel Trench with Pipes Results ............. 122
  7.5 Gravel Trench with Pipes and Source Zone Pumping Results 125
  7.6 Comparison ...................................... 128

8 Fractured Bedrock Scenario ....................... 130
  8.1 Groundwater Flow Calibration ................... 134
  8.2 Contaminant Transport ........................ 138
  8.3 Gravel Trench Results .......................... 141
  8.4 Gravel Trench with Pipe Results .............. 144
  8.5 Gravel Trench with Pipe and Source Zone Pumping Results 146
  8.6 Comparison ...................................... 148

9 Heterogeneous Intact Bedrock Scenario .......... 150
  9.1 Groundwater Flow Calibration ................... 153
  9.2 Gravel Trench Results .......................... 158
  9.3 Gravel Trench with Pipe Results .............. 162
List of Tables

1.1 Different hydraulic isolation designs considered for buried waste in Oak Ridge, TN. Adapted from Davis and Stansfield (1984) . . . . . . . . 8
3.1 Dimensions and parameters of the conceptual aquifer model. . . . . . . . . 25
3.2 Dimensions and properties of the gravel trench design. . . . . . . . . 27
3.3 Summary of the properties for the transport simulation . . . . . . . . 30
4.1 Parameters used to define the Markov chains in T-Progs . . . . . . . . 58
4.2 Relevant flow parameters for the T-Progs generated material distribution. 60
4.3 Dimensions of the hydraulic barrier designs considered in this chapter. 62
4.4 Summary of the properties of the transport simulation . . . . . . . . 63
List of Figures

1.1 A variety of groundwater remediation strategies using Pump and Treat or in conjunction with Pump and Treat and their idealized performance. Adapted from Cohen et al. (1997). 4
1.2 The life cycle and evolution of groundwater remediation strategies over time. Adapted from Ellis and Hadley (2009). 5
1.3 Hydraulic isolation flow field for a waste disposal site. Adapted from Zijl and Nawalany (1993). 10
1.4 Different hydraulic isolation schematics that include treatment of contaminated water. Adapted from Zijl and Nawalany (1993). 11
1.5 Schematic of the Hydrafaraday passive hydraulic isolation design. Adapted from Vigouroux et al. (2015). 12
1.6 Schematics of the Ankeny moat. Adapted from Ankeny and Forbes (1997). 13
1.7 Map of the location where the Ankeny moat was installed. The green represents the French drains used to hydraulically isolate the source zone. Adapted from USEPA (2004). 14

2.1 A gravel trench hydraulic barrier. 17
2.2 A hydraulic barrier that consists of a gravel trench and pipe. 20
2.3 A hydraulic isolation system that consists of a gravel trench with pipe and injection and extraction wells. 21
2.4 A hydraulic isolation system design that includes source zone pumping in addition to a gravel trench with pipe. 22

3.1 The grid and model boundary conditions for the conceptual model. A constant head of 15.2 m is assigned to the upgradient boundary and 14.9 m to the downgradient boundary. The remaining sides are no flow boundaries. 24
3.2 Groundwater simulation results for the conceptual model under natural conditions. 26
3.3 Hydraulic barrier inclusion in conceptual model simulations. The units for the horizontal hydraulic conductivity scale are in m/d. 27
3.4 Groundwater flow simulation results for the conceptual model upon inclusion of a gravel trench hydraulic barrier (black). 28
3.5 The location of the specified concentration tracer used to represent a source zone in relation to the hydraulic barrier (red). ............................ 29
3.6 A depiction of how the source zone and the plume are differentiated for plume mass calculations. Only cells outside of the source zone will be used to calculate plume mass. ............................................. 31
3.7 Tracer concentrations after 30 years for different effective hydraulic conductivity values. The results show a k=12 plan view. The units for the scale are mg/L. ................................................................. 33
3.8 Tracer concentrations after 30 years for different trench hydraulic conductivity values. The results are shown for the cross section i=50. The units for the scale are mg/L. (Z magnification = 3) ....................... 34
3.9 Normalized plume mass as a function of time for different trench effective hydraulic conductivity values. The hydraulic conductivity of the aquifer is 8.53 m/d. ................................................................. 36
3.10 Tracer concentrations after 30 years for different gravel trench (K=853 m/d) depth designs. The results show a k=12 plan view. The units for the scale are mg/L. ................................................................. 38
3.11 Tracer concentrations after 30 years for different gravel trench (K=853 m/d) depth designs. The results are shown for the cross section i=50. The units for the scale are mg/L. (Z magnification = 3) ....................... 39
3.12 Normalized plume mass as a function of time for gravel trenches (K=853 m/d) installed to different depths. ................................. 41
3.13 Tracer concentrations after 30 years for different gravel trench (K=853 m/d) thickness designs. The results show a k=12 plan view. The units for the scale are mg/L. ................................................................. 43
3.14 Tracer concentrations after 30 years for different gravel trench (K=853 m/d) thickness designs. The results are shown for the cross section i=50. The units for the scale are mg/L. (Z magnification = 3) ....................... 44
3.15 Normalized plume mass as a function of time for gravel trenches (K=853 m/d) constructed with different thicknesses. ...................... 45
3.16 Top view of the location of pumping and injection wells (yellow) in relation to the gravel trench hydraulic barrier. The magnitude of the volumetric flow rate (Q) for each well is 5.4 L/min. ...................... 46
3.17 Plume concentrations at selected times after inclusion of a gravel trench with pumping and injection wells. The results shown are for a plan view of k=12. The units for the scale are mg/L. ...................................... 47
3.18 Plume concentrations at selected times after inclusion of a gravel trench with pumping and injection wells. The results are for the cross section i=50. The units for the scale are mg/L. (Z magnification=3) ....................... 48
3.19 Contaminant transport simulation results for different hydraulic barrier designs after 30 years of simulation. The results are in plan view for k=12. The units for the scale are mg/L. ...................................... 50
3.20 Contaminant transport simulation results for different hydraulic barrier designs after 30 years of simulation. The results are for the cross section i=50. The units for the scale are mg/L. (Z magnification=3) 51

3.21 Normalized plume mass as a function of time comparing a gravel trench design with a gravel trench with wells design. 52

3.22 Plume concentrations after 30 years of simulation for a gravel trench with pipe and pumping and injection wells that penetrate an aquifer to a depth of 12.2 m. The units for concentration are in mg/L. 53

3.23 Normalized plume mass as a function of time comparing a fully penetrating gravel trench, fully penetrating gravel trench with wells design, and a partially penetrating gravel trench with wells design. 54

4.1 Schematic representation of a transition probability. Adapted from Carle (1999). 56

4.2 Graphical representation of TPROGS parameters. Adapted from Aqua-veo (2017). 57

4.3 Markov Chains in the Z-direction using the parameters specified in Table 4.1. 58

4.4 Heterogeneous spatial distribution for a conceptual model with two materials. (Z magnification=3) 59

4.5 Groundwater simulation results for the heterogeneous conceptual model under natural conditions. 61

4.6 Hydraulic barrier inclusion by adding a new material and the location of the specified tracer concentration. 62

4.7 The selected area (all cells outside of the source zone) defined as the plume and used to calculate the plume mass is highlighted. The isolated area within the hydraulic barrier is considered the source zone and not included in plume mass calculations. Equation 3.1 is used to calculate plume mass. 63

4.8 The base case plume concentrations after 30 years for the heterogeneous conceptual model. 64

4.9 Plume mass as a function of time for the base case heterogeneous conceptual model plume shown in Figure 4.8. 65

4.10 Tracer concentrations for different values of trench hydraulic conductivity after 30 years of simulation. The results are for the k=12 plan view. The units for the scale are mg/L. 67

4.11 Tracer concentrations for different values of trench hydraulic conductivity after 30 years of simulation. The cross sections are for i=50. The units for the scale are mg/L. (Z magnification =3) 68

4.12 Normalized plume mass as a function of time for a heterogeneous conceptual model for different hydraulic barrier hydraulic conductivity values. 69
4.13 Tracer concentrations for different values of trench depth after 30 years of simulation. The results are for the k=12 plan view. The unit for concentration is mg/L. ....................................................... 71
4.14 Tracer concentrations for different values of trench depth after 30 years of simulation. The cross sections are for i=50. The unit for concentration is mg/L. (Z magnification =3) ....................................................... 72
4.15 Normalized plume mass as a function of time for a heterogeneous conceptual model for different hydraulic barrier depth designs. .......... 74
4.16 Tracer concentrations for different values of trench thickness after 30 years of simulation. The results are for the k=12 plan view. The unit for concentration is mg/L. ....................................................... 75
4.17 Tracer concentrations for different values of trench thickness after 30 years of simulation. The cross sections are for i=50. The unit for concentration is mg/L. (Z magnification =3) ....................................................... 76
4.18 Normalized plume mass as a function of time for a heterogeneous conceptual model for different hydraulic barrier thickness values. ...... 77

5.1 Map of the Barra Mansa site with the area of focus boxed in red, the Goiabal stream in blue, and the Paraiba river labeled. ............... 79
5.2 Barra Mansa site with local stream and contours of hydraulic head from 2004. The 378 m head contour is outlined in bold blue. (DuPont, 2016) ................................................................. 80
5.3 The location and geometry of the model boundary conditions: the general head boundary (red), the river boundary (blue), and the constant head boundary (orange). The length along the stream is 320 m for reference. ....................................................... 82
5.4 Regional geologic map of the site of interest. (Ch2m, 2017) ............... 84
5.5 Calibrated recharge polygons and associated values. The length along the stream is 320 m for reference. ....................................................... 86
5.6 Schematic depicting the interpretation of the error bars used for model calibration ....................................................... 87
5.7 An areal view of the entire grid after refinement. The length of the model along the stream is 320 m for reference. ......................... 89
5.8 An enlarged view of the grid refinement points in the area where the source zone and hydraulic barrier design occurs. The dimensions of the refined rectangle are 53 m x 38 m ......................... 90
5.9 Concentration contour map from 2004 with legend for chloroform at the site. The highest concentration contour on the map is used to create the areal extent of the idealized source zone used in this work. (DuPont, 2016) ......................... 91
5.10 The source zone is depicted and indicated by the arrow as the purple region. The source zone is assigned a constant concentration of 10,000 ppb.

5.11 An aerial view of the source zone (pink) and the hydraulic barrier (red) that surrounds it.

5.12 The fully penetrating hydraulic barrier inclusion for the simulations done in the upcoming chapters.

5.13 The hydraulic barrier and a plan view of the location of the source zone pumping well (white circle).

6.1 Calibrated hydraulic conductivity polygons for the intact bedrock case. The hydraulic conductivities shown here correspond to aquifer layers 1-7 of the model. The anisotropy of the aquifer is 10. The units for the legend are m/d. The length along the stream is 320 m for scale.

6.2 The hydraulic conductivity of the bedrock is $8 \times 10^{-4}$ m/d. The bedrock is treated as being isotropic and the units for the legend are m/d. The length along the stream is 320 m for scale.

6.3 The groundwater head contours and model calibration results for the intact bedrock scenario. The calibration targets are generally met throughout the model. The unit for the legend is meters. The length along the stream is 320 m for scale.

6.4 An enlarged view of the error bars in the focus area for this analysis with the intact bedrock scenario results. All but two of the twenty-nine calibration target intervals in this area are met. The unit for the legend is meters.

6.5 Computed heads plotted against observed heads for the intact bedrock scenario. The units for the heads in both axes is meters.

6.6 Base case plume concentrations after 30 years of simulation for the intact bedrock scenario. The unit for concentration is parts per billion (ppb). The length along the stream is 320 m for scale.

6.7 Plume mass as a function of time for the base case condition of the intact bedrock scenario.

6.8 Contaminant concentrations after 60 years of simulation with the inclusion of a gravel trench design for the intact bedrock scenario. The units for concentration are ppb. The length along the stream is 320 m for scale.

6.9 Plume mass as a function of time comparing the plume mass without a hydraulic barrier (blue) to the plume mass when a gravel trench is included after 30 years (orange).

6.10 Contaminant concentrations after 60 years of simulation with the inclusion of a gravel trench with pipe design for the intact bedrock scenario. The units for concentration is ppb.
<table>
<thead>
<tr>
<th>Number</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.11</td>
<td>Plume mass as a function of time comparing the plume mass without a hydraulic barrier (blue) to the plume mass when a gravel trench with pipe is included after 30 years (orange).</td>
</tr>
<tr>
<td>6.12</td>
<td>Comparison of plume mass over time for the gravel trench (green) and gravel trench with pipe (blue) designs for the intact bedrock scenario.</td>
</tr>
<tr>
<td>7.1</td>
<td>Hydraulic conductivity calibration polygons used for the aquifer layers 1-3 and 5-6 for the highly conductive layer scenario. The anisotropy of each of the aquifer layers is 10. The units for the scale are m/d. The length along the stream is 320 m for scale.</td>
</tr>
<tr>
<td>7.2</td>
<td>The hydraulic conductivity of the highly conductive bed at 3 m/d. The layer has an anisotropy of 10 and the units for the scale are m/d. The length along the stream is 320 m for scale.</td>
</tr>
<tr>
<td>7.3</td>
<td>The hydraulic conductivity of the bedrock as $8 \times 10^{-4}$ m/d. The bedrock is treated as being isotropic and the units for the scale are m/d. The length along the stream is 320 m for scale.</td>
</tr>
<tr>
<td>7.4</td>
<td>The groundwater head contours and model calibration results for the highly conductive layer scenario. Thirty out of the thirty five wells used for model calibration meet the calibration target interval. The scale is in meters. The length along the stream is 320 m for scale.</td>
</tr>
<tr>
<td>7.5</td>
<td>The groundwater head contours and error bars where the source zone and hydraulic barrier will be included for the highly conductive layer scenario. The scale is in meters.</td>
</tr>
<tr>
<td>7.6</td>
<td>Plot of the computed heads vs the observed heads for the highly conductive layer scenario. Both axes have units of meters.</td>
</tr>
<tr>
<td>7.7</td>
<td>Base case contaminant concentrations results after 30 years of simulation for the highly conductive layer scenario. The scale has units of ppb. The length along the stream is 320 m for scale.</td>
</tr>
<tr>
<td>7.8</td>
<td>Plume mass as a function of time for the base case highly conductive layer scenario.</td>
</tr>
<tr>
<td>7.9</td>
<td>Concentration distribution after 60 years upon inclusion of the gravel trench for the highly conductive layer scenario. The concentrations are in units of ppb. The length along the stream is 320 m for scale.</td>
</tr>
<tr>
<td>7.10</td>
<td>Plume mass as a function of time comparing the plume mass without a hydraulic barrier (blue) with the plume mass if a gravel trench (orange) is included after 30 years.</td>
</tr>
<tr>
<td>7.11</td>
<td>Concentration distribution after 60 years of simulation after inclusion of the gravel trench with pipe design for the highly conductive layer scenario. The units for concentration are in ppb. The length along the stream is 320 m for scale.</td>
</tr>
</tbody>
</table>
7.12 Plume mass as a function of time comparing the plume mass without a hydraulic barrier (blue) with the plume mass if a gravel trench with pipe (orange) is included after 30 years ........................................ 125
7.13 Concentration distribution after 60 years upon inclusion of the gravel trench with pipe and source zone pumping design for the highly conductive layer scenario. The units for concentration are ppb. The length along the stream is 320 m for scale. ........................................ 126
7.14 Plume mass as a function of time comparing the plume mass without a hydraulic barrier (blue) with the plume mass if a gravel trench with pipe and source zone pumping (orange) is included after 30 years. . 127
7.15 Plume mass as a function of time for the inclusion of a gravel trench (green), gravel trench with pipe (blue), and source zone pumping (yellow). 129
8.1 Calibrated hydraulic conductivity polygons for aquifer layers 1-6 of the fractured bedrock case. The anisotropy of each of the aquifer is 10. The units for the scale are m/d. The length along the stream is 320 m for reference. ........................................ 131
8.2 The calibrated hydraulic conductivity polygons for the fractured bedrock aquifer layer 7. The fractured bedrock is assumed to be isotropic. The units for the scale are m/d. The length along the stream is 320 m for reference. ........................................ 132
8.3 The hydraulic conductivity of the bedrock as $8 \times 10^{-4}$ m/d. The bedrock is treated as being isotropic and the units for the scale are m/d. The length along the stream is 320 m for reference. ......................... 133
8.4 The groundwater head contours and model calibration results for the fractured bedrock scenario. The unit for the legend is meters. The length along the stream is 320 m for scale. ........................................ 135
8.5 The groundwater head contours and error bars where the source zone and hydraulic barrier will be included for the fractured bedrock scenario. The unit for the legend is meters. ......................... 136
8.6 Plot of the computed heads vs the observed heads for the fractured bedrock scenario. The units for head along both axes is meters. . . . . 137
8.7 Base case tracer concentrations after 30 years for the fractured bedrock scenario. The units for the legend are in ppb. The length along the stream is 320 m for scale. ................................. 139
8.8 Plume mass as a function of time for the fractured bedrock base case. 140
8.9 Tracer concentrations after 60 years of simulation with the inclusion of a gravel trench for the fractured bedrock scenario. The units for the legend are in ppb. The length along the stream is 320 m for scale. . . . . 142
8.10 Plume mass as a function of time comparing the plume mass without a hydraulic barrier (blue) with the plume mass if a gravel trench is included after 30 years (orange) for the fractured bedrock scenario. . 143
8.11 Tracer concentrations after 60 years upon inclusion of the gravel trench with pipe design for the fractured bedrock scenario. The units for the legend are in ppb. The length along the stream is 320 m for scale.

8.12 Plume mass as a function of time comparing the plume mass without a hydraulic barrier (blue) with the plume mass if a gravel trench with pipe is included after 30 years (orange) for the fractured bedrock scenario.

8.13 Tracer concentrations after 60 years upon inclusion of the gravel trench with pipe and source zone pumping design for the fractured bedrock scenario. The units for the legend are in ppb. The length along the stream is 320 m for scale.

8.14 Plume mass as a function of time comparing the plume mass without a hydraulic barrier (blue) with the plume mass if a gravel trench with pipe and source zone pumping is included after 30 years (orange) for the fractured bedrock scenario.

8.15 Plume mass as a function of time with the inclusion of a gravel trench (blue), gravel trench with pipe (orange), and source zone pumping (gray).

9.1 Example of a borehole from the site

9.2 Spatial locations of borehole logs

9.3 Heterogeneous intact bedrock scenario spatial distribution of materials

9.4 Calibrated material properties for heterogeneous intact bedrock scenario

9.5 The groundwater head contours and model calibration results for the heterogeneous intact bedrock scenario. The heads are measured in meters and the distance along the stream is 320 m for scale.

9.6 The groundwater head contours and calibration error bars zoomed into the area of greatest importance in the model for the heterogeneous intact bedrock scenario. The heads are in meters.

9.7 Plot of the computed heads vs observed heads for the heterogeneous intact bedrock scenario. The units along both of the axes is meters.

9.8 Concentration distribution after 30 years of transport for the base case heterogeneous intact bedrock scenario. The concentration units are in ppb and the distance along the stream is 320 m for scale.

9.9 Concentration distribution after 60 years of simulation upon inclusion of the gravel trench (K=853 m/d) for the heterogeneous intact bedrock scenario. The concentration units are in ppb and the stream has a length of 320 m for scale.

9.10 Plume mass as a function of time for the case where no hydraulic barrier is installed (blue) and a case where inclusion of a gravel trench (K=853 m/d) occurs after 30 years (orange).
9.11 Concentration distribution after 60 years of simulation upon inclusion of the gravel trench with pipe (K=8530 m/d) for the heterogeneous intact bedrock scenario. The concentration units are in ppb and the stream has a length of 320 m for scale.

9.12 Plume mass as a function of time for the case where no hydraulic barrier is installed (blue) and a case where inclusion of a gravel trench with pipe occurs after 30 years (orange).

9.13 Concentration distribution after 60 years upon inclusion of the gravel trench with pipe (K=8530 m/d) and a low flow source zone pumping well (Q=0.278 L/min) for the heterogeneous intact bedrock scenario.

9.14 Plume mass over time with and without the inclusion of a gravel trench with pipe and source zone pumping for the heterogeneous intact bedrock scenario.

9.15 Plume mass reduction compared for the gravel trench (orange), gravel trench with pipe (yellow), and gravel trench with pipe and source zone pumping (green) designs for the heterogeneous intact bedrock scenario.

10.1 Calibrated material properties for the heterogeneous fractured bedrock scenario.

10.2 Spatial distribution of materials for the heterogeneous fractured bedrock scenario.

10.3 The groundwater head contour results and error bars for the calibration targets. The units of the scale are in meters and the length along the stream is 320 m for reference.

10.4 The groundwater head contours and calibration error bars zoomed into the area of greatest importance for the heterogeneous fractured bedrock scenario. The unit for head is meters.

10.5 Plot of the computed heads vs observed heads for the heterogeneous fractured bedrock scenario. The unit for both of the axes is meters.

10.6 Concentration distribution after 30 years of transport for the base case heterogeneous fractured bedrock scenario. The units for concentration are in ppb and the length along the stream is 320 m for scale.

10.7 Concentration distribution after 60 years upon inclusion of the gravel trench (K=853 m/d) for the heterogeneous fractured bedrock scenario. The units for concentration are in ppb and the length along the stream is 320 m for scale.

10.8 Plume mass as a function of time for the case where no hydraulic barrier is installed (blue) and a case where inclusion of a gravel trench (K=853 m/d) occurs after 30 years (orange).
10.9 Contaminant distribution after 60 years for the gravel trench with pipe (K=8,530 m/d) design for the heterogeneous fractured bedrock scenario. The units for concentration are in ppb and the length along the stream is 320 m for scale.

10.10 Plume mass as a function of time for the case where no hydraulic barrier is installed (blue) and a case where inclusion of a gravel trench with pipe (K=8,530 m/d) occurs after 30 years (orange).

10.11 Concentration distribution after 60 years upon inclusion of the gravel trench with pipe (K=8,530 m/d) and a low flow source zone pumping well for the heterogeneous fractured bedrock scenario. The units for concentration are in ppb and the length along the stream is 320 m for scale.

10.12 Plume mass as a function of time without the inclusion of a hydraulic barrier (blue) compared to a case where the gravel trench with pipe and source zone pumping (orange) is included after 30 years for the heterogeneous fractured bedrock scenario.

10.13 Plume mass as a function of time compared for the gravel trench (blue), gravel trench with pipe (green), and gravel trench with pipe and source zone pumping (yellow) designs for the heterogeneous fractured bedrock scenario.

1 The dimensions of the 2D trench are 4.4 meters in width and 10 meters in length. Three pipes (smaller rectangles) are included in this trench and each pipe has a diameter of 10 centimeters.

2 Numerical simulation of parallel flow in pipe and porous media. The scale has units of m/s.
Chapter 1

Introduction

1.1 Emergence of Groundwater Contamination

Prior to the Industrial Revolution, the primary threat to groundwater quality around human communities was from biological agents (Matthews et al., 2008). These substances, when subjected to filtration, microbial degradation, redox reactions, adsorption, and other natural phenomena had a temporary presence in the subsurface. The introduction of chemicals that were far more resilient to natural processes largely began during the Industrial Revolution (Rail, 2000). Poor and unregulated waste disposal techniques were practiced under the guise that the subsurface was a natural filter (Mutch and Eckenfelder, 1993; NRC, 1994). Large scale development of synthetic organics such as chlorinated solvents, herbicides, and pesticides and many other chemicals resulted in an even more widespread and persistent groundwater contamination problem. Although some professionals at the time knew and expressed that soil had a limited ability to degrade waste and that contaminants can travel long distances (Colten, 1998), it was not until the late 1970s that changes in legislature were made to address the issue. These changes included passage of the
Resource Conservation and Recovery Act (RECRA) and Toxic Substances Control Act (TSCA) to increase regulation of industrial waste practices. In 1980, the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA), saw large scale groundwater remediation efforts get underway with the implementation of the EPA Superfund remediation program. However, the implementation of legislation such as RECRA, TSCA, and CERCLA happened after most of today’s problems were created (Pankow and Cherry, 1996; Kueper et al., 2014).

1.2 Groundwater Remediation Challenges

Groundwater remediation has proven to be a challenging task. Returning contaminated groundwater to meet stringent maximum contaminant levels (MCL) and drinking water standards requires stopping a contaminant plume from advancing, either immobilizing, removing, or destroying contaminant mass in the source area, and treating the plume. The conventional pump and treat method, where contaminated groundwater is extracted, treated at the surface, and reinjected into the subsurface, was once seen as a promising remedy. However, time has shown that although the initial decrease in the extracted waters contaminant concentration can be rapid, it then begins to tail off and often fails to meet the MCL (Mackay and Cherry, 1989; NRC, 1994).

Many processes, including desorption of the contaminant, matrix diffusion, precipitate dissolution, and groundwater velocity variations result in tailing and rebounding of the contaminant concentration in the extracted groundwater, make it nearly impossible to reduce contaminant concentrations below the drinking water standard within a reasonable time frame (Mackay and Cherry, 1989; NRC, 1994). An adaptation of the pump and treat method is plume containment (Cohen et al.,
The migration of contaminant plumes offsite is a threat to human exposure and by withdrawing groundwater and reversing the hydraulic gradient, a plume can potentially be contained. Containing a plume typically requires lower pumping rates than the use of the conventional pump and treat method. The drawback to plume containment is that the pumping system is in operation indefinitely. Additionally, small-scale heterogeneities can have a significant impact on the effectiveness of hydraulic containment. Preferential flow paths can result in zones that are depleted of a contaminant to be pumped while zones that are still highly contaminated to be unaffected.

Despite the limitations of the performance of Pump and Treat alone, if it is used in combination with other designs it can be beneficial. Figure 1.1 shows some different groundwater remediation designs that use Pump and Treat. In addition to plume containment, Pump and Treat can be used to cut off an existing plume or treat a plume downgradient of a hydraulic containment well. A cut-off wall can also be used for containment, instead of a pumping well, and Pump and Treat used to clean up a plume. Similarly, the source zone can be removed and the downgradient plume treated to achieve aquifer restoration. The depictions in Figure 1.1 exhibit that unless additional technologies are implemented to contain or remove the source zone, the remediation effort may be in operation virtually indefinitely.
1.3 Sustainable Remediation

Due to the challenges associated with remediating contaminated sites, alternative methods are sought that employ more sustainable technologies. Energy intensive groundwater remediation strategies, can not only be costly but are also not guaranteed to meet clean up goals (NRC, 2005). The carbon footprint for some of these processes can be quite large as well, affecting their overall contributions to environmental restoration (Ellis and Hadley, 2009; USEPA, 2009). Other potential environmental side-effects that can be introduced by an environmental remediation operation may include waste production, ecological impact, and noise. Figure 1.2 shows the evolution of different approaches towards groundwater remediation over time and captures the transition to more sustainable environmental solutions. The
motivation behind sustainable remediation is to provide a net benefit to the environment by efficiently managing limited resources (Ellis and Hadley, 2009). Equipment such as trucks, pumps, and soil vapor extraction systems to name a few, should be used in good judgment (USEPA, 2008). At times, a combination of remediation techniques may be necessary to achieve remediation targets at a lower cost and with fewer secondary environmental impacts than alternatives.

Figure 1.2: The life cycle and evolution of groundwater remediation strategies over time. Adapted from Ellis and Hadley (2009).

1.4 Hydraulic Isolation

Hydraulically isolating or containing a contaminant source may provide a sustainable, alternative approach to groundwater remediation. Contact between clean groundwater and a contaminant source can be minimized or eliminated if hydraulic isolation is achieved. Treating contaminant source zones is difficult and in the case
of DNAPLs, very complex due to their ability to partition into different phases, low solubility, and persistence in the environment (Kueper et al., 2014). Hydraulic isolation would allow for greater contact time for in-situ treatments, such as oxidation or bioremediation, which could improve the performance of source zone treatments, while also accelerating the natural attenuation of an existing plume. There are two approaches to hydraulic isolation: active isolation and passive isolation.

Active hydraulic isolation can be characterized by the inclusion of wells and/or contaminated water treatment for containment, whereas passive hydraulic isolation utilizes material of contrasting permeability for the same purposes (NRC, 2012). Active hydraulic isolation for plume containment or isolation is well established and commonly practiced. The drawbacks include energy consumption, difficulties in designing an effective system, and cost of treatment or disposal of contaminated water. Ultimately, active isolation technologies involve technical operations that are required continuously over time (Nawalany, 1995). Additionally, the design of an adequate active hydraulic isolation system is dependent on the site characterization data available and for highly heterogeneous sites, active designs that include wells may need to be pumped aggressively to effectively contain a plume (Cohen et al., 1997).

Remediation systems that do not require the use of wells and minimize energy consumption, yet are able to achieve the remediation goals of a project are ideal but uncommon. Two approaches can be used to achieve passive isolation. The construction of an impermeable barrier, such as a sheet pile or slurry wall, is one approach that can be used to achieve containment of a source zone (NRC, 2007). This would provide a design for a hydraulic barrier through material that has a hydraulic conductivity that is many orders of magnitude less than the aquifer material. However impermeable barriers typically need to be excavated to bedrock and need to be constructed to near perfection to be effective. Impermeable barriers would also cause a rise in the
upstream hydraulic head and create a scenario where groundwater mounding takes place.

Permeable hydraulic barriers, such as French drains (Davis and Stansfield, 1984) or gravel trenches (Ankeny and Forbes, 1997), provide another approach for source zone containment. Permeable hydraulic barriers could offer a cheaper design that is more robust to small defects than impermeable barriers. A permeable hydraulic barrier would act nearly as an equipotential, so groundwater mounding may not be a concern. Therefore, considering the cost of construction, durability, and the fact that a permeable hydraulic barrier could easily be monitored with monitoring wells, it may be a viable option for hydraulic source zone isolation and will be the focus of this work.

1.5 Prior Work

One of the earliest published implementations of a permeable hydraulic barrier was described by Davis and Stansfield (1984). Buried drains would become saturated due to seasonal changes in the water level resulting in the mobilization of Strontium 90 in Oak Ridge, Tennessee. After considering a variety of means to isolate the buried waste, a passive design was selected. A table listing the different designs considered for the Oak Ridge project is summarized in Table 1.1. The narrow French drain (Scheme Ia) was selected due to many advantages it offered over the other designs considered.

The width of the French drain was 1 meter, which was the same width of the trench, and 15.24 cm (6 inch) diameter pipe was used. The trench was excavated to a maximum depth of 10 meters. The French drain was meant to intercept upstream groundwater before it could reach the disposal trenches. The trench was back-filled
Table 1.1: Different hydraulic isolation designs considered for buried waste in Oak Ridge, TN. Adapted from Davis and Stansfield (1984).

<table>
<thead>
<tr>
<th>Scheme</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Drains</strong></td>
<td></td>
</tr>
<tr>
<td>Scheme Ia</td>
<td>A French drain in a narrow trench excavated with a backhoe</td>
</tr>
<tr>
<td>Scheme Ib</td>
<td>A French drain with sloped excavation 1.5:1 slope</td>
</tr>
<tr>
<td>Scheme Ic</td>
<td>A French drain installed in a braced excavation</td>
</tr>
<tr>
<td>Scheme Id</td>
<td>A free draining rock wall constructed with drilled piers</td>
</tr>
<tr>
<td><strong>Barriers</strong></td>
<td></td>
</tr>
<tr>
<td>Scheme IIa</td>
<td>A narrow slurry trench excavated with a backhoe</td>
</tr>
<tr>
<td>Scheme IIb</td>
<td>A cementitious cutoff wall constructed with drilled piers</td>
</tr>
<tr>
<td>Scheme IIc</td>
<td>A sheet pile cutoff wall</td>
</tr>
<tr>
<td><strong>Combination of Drain and Barrier</strong></td>
<td></td>
</tr>
<tr>
<td>Scheme IIIa</td>
<td>A French drain in series with a slurry trench (Schemes Ia and IIa)</td>
</tr>
<tr>
<td>Scheme IIIb</td>
<td>A French drain with an impervious membrane included and excavated with a backhoe</td>
</tr>
<tr>
<td>Scheme IIIc</td>
<td>A cementitious cutoff wall constructed with drilled piers and a free draining rock wall (Schemes Id and IIb)</td>
</tr>
</tbody>
</table>
with crushed stone and capped with bentonite. Additionally, the French drain acted to lower the hydraulic head below the bottom of waste-filled trenches. Initially, after installation of the French drain, the groundwater table was below over 50% of the burial site at the targeted area and five trenches were completely dewatered. However, additional investigations later revealed that the bentonite cap was in fact leaky which worked against the goal of the French drain. Nevertheless, the French drain proved to be an economical and effective means for reducing the risk of exposure. No additional need for maintenance or remediation for the installation was required after installation and it showed great promise. Davis and Stansfield (1984a) provides a good outline of the entire process of selection, design, construction, and monitoring of a passive hydraulic isolation design.

Zijl and Nawalany (1993) include an example of a finite element numerical model for hydraulic isolation of a waste disposal site. The hydraulic conductivity in the model is assumed to be isotropic in the x and y directions (perfect layering) and variable in the z direction. The design consists of two, shallow and parallel ditches. A scenario where the head difference between the two shallow ditches are too small to isolate the waste disposal site from the regional groundwater flow system is shown in Figure 1.3a. If the head difference between the two ditches is too small, then the streamlines originating from the waste disposal site join the groundwater flow field. If a sufficient head difference is maintained between the two ditches, then a local flow system is created and the waste disposal site is effectively isolated (Figure 1.3b). The head differences between the two trenches would need to be maintained and that would likely require some type of pumping of the contaminated water in the downstream ditch, treatment of the contaminated water, and addition of the treated water to the upstream ditch. Additional factors that affect the performance of the hydraulic isolation design are the resistivity of the bottom of the trench and the
distance between the two trenches in Figure 1.3.

(a) Incomplete hydraulic isolation of a waste disposal site using ditches (b) Complete hydraulic isolation of a waste disposal site using ditches

Figure 1.3: Hydraulic isolation flow field for a waste disposal site. Adapted from Zijl and Nawalany (1993).

Nawalany (1995) introduced schematics for the design of hydraulic isolation systems that use wells and drains in addition to the ditch design shown in Figure 1.3. These designs are proposed to make a case for hydraulically isolating waste disposal sites. One of the key features of these hydraulic isolation techniques that is emphasized is the zero local water balance so that undesired side-effects such as groundwater depletion and land subsidence can be avoided. All of the designs discussed in (Nawalany, 1995) are active technologies and require the treatment of contaminated water. The schematic shown in Figure 1.4a exhibits pumped water being treated and treated wastewater being reinjected into the subsurface. The use of ditches with the same design as Zijl and Nawalany (1993) is also revisited. Ditches or trenches provide one of the simplest means of hydraulic isolation but the designs that are shown are not passive. A difference in hydraulic head needs to be maintained through pumping and treatment of the contaminated water collected in a downstream ditch and and recharging the upstream ditch with the treated water. Although clogging of the base of the ditch is a concern and can significantly reduce the effectiveness of local isolation of the groundwater system, the attraction to such technology is its simplicity.
Recharge and abstraction drains are also considered (Figure 1.4c). The recharge and abstraction drains need to be located close to the bottom and cover the planar extent of the waste disposal site. The disadvantage of using drains for hydraulic isolation is that they may only be useful for small plumes. Contaminant plumes of regional extent may be remediated with designs that include wells and ditches.

(a) Wells used to achieve hydraulic isolation.  (b) Ditches used to achieve hydraulic isolation.  (c) Drains used to achieve hydraulic isolation.

Figure 1.4: Different hydraulic isolation schematics that include treatment of contaminated water. Adapted from Zijl and Nawalany (1993)

Vigouroux et al. (2015) used FEFLOW (Diersch, 2009) to model a passive hydraulic isolation system. The motivation behind this work was to improve the effectiveness of a chemical treatment on a contaminated aquifer in France. For this to be effective, the contact time of the groundwater and the prescribed chemicals has to be increased. The suggested design consisted of horizontal drains connected to an upstream and downstream well. The upstream and downstream wells are connected to each other with a single pipe. The design shown in Figure 1.5 displays the schematics of the passive containment design. It is a modification of a patented design called HydrauFarady (Boisson et al., 2002). The site is an active industrial site so large scale excavations were unattractive due to the level of industrial activity and possibility of disturbing traffic at the site. Thus, a trenchless method was selected and drilling minimized in its construction. Initial simulations showed flow taking place through the source zone to the drains, which suggested that the drains were not deep enough. Node averaging of groundwater velocity through the source zone was used to quantify
the flow through the source zone and optimize the design of the passive isolation system by comparing the performance of the passive hydraulic isolation system design to a scenario where no such system is included. The two final designs that were suggested were able to reduce the groundwater velocity through the source zone by 70% and 90% respectively.

Figure 1.5: Schematic of the HydauFaraday passive hydraulic isolation design. Adapted from Vigouroux et al. (2015).

Ankeny and Forbes (1997) obtained a patent (USA Patent Number 6139221) on constructing a constant hydraulic head moat. It is a simple and versatile concept that involves digging a trench, burying pipe at the bottom, and back-filling the trench with gravel. The result is a significant reduction in the head gradient through the isolated zone, either eliminating or greatly reducing flow through the contaminant source zone. Suggestions are also made for the addition of trees, plants, or mechanized pumps in the source zone to create an upward and inward gradient in the source area. If source zone isolation can be achieved without mechanized pumping, then the hydraulic isolation installation would provide a long term, passive design.

The design proposed in Ankeny and Forbes (1997) is environmentally friendly, robust, economical, and low maintenance. These benefits led to an EPA implementation of what’s been termed the "Ankeny Moat" (USEPA, 2004). The Ankeny moat
was installed to isolate a shallow perched aquifer at a site where BTEX was leaking from a gas station. A drainage system was installed around the source zone. Horizontal drilling technology was used to install the drains so excavation of a trench was not needed. The drains were made with porous polyethylene pipe with a pore size of 40 microns. Risers were installed as monitoring wells to measure the hydraulic head in different sections of the drain. The risers allowed assessments to be made of the hydraulic connectivity of the drain. The moat also served a dual purpose in delivering upstream nutrients to assist in the natural attenuation of the downstream plume. Preliminary results indicated that the water levels in the risers stabilized and the hydraulic gradient in the source zone was reduced by 50%. Geochemical monitoring was performed and the number of electron acceptors were greater than the number of electron donors in the southeast riser. This was attributed to the moat bringing in sulfate from upgradient of the source. In the vicinity of the source zone however the situation was reversed. These results suggest that natural attenuation of the plume may have been enhanced and the flow through the source zone reduced, however, the author could not find any additional information on the long term performance of this system.
1.6 Research Goals

The goal of this thesis is to analyze the performance of permeable hydraulic barrier designs under different hydrogeologic conditions using MODFLOW (McDonald and Harbaugh, 1984) and MT3DMS (Zheng and Wang, 1999). Various hydraulic barrier designs based on the Ankeny moat concept are considered in this work. The strengths and weaknesses of a passive hydraulic isolation system are explored and suggestions for potential field applications provided. This goal is achieved by:

- Considering different hydraulic source zone isolation designs (Chapter 2) and demonstrating how hydraulic source zone isolation alters the head gradient in a homogeneous conceptual aquifer model (Chapter 3)
- Performing a sensitivity analysis of trench design parameters on a homogeneous
conceptual aquifer model (Chapter 3) and a heterogeneous conceptual aquifer model (Chapter 4) using contaminant transport simulations that consider a continuous source that generates a biodegradable plume

• Constructing a numerical groundwater and contaminant transport model of a prospective site that considers a continuous source that produces a conservative plume (Chapter 5)

• Calibrating groundwater flow models and running contaminant transport simulations for scenarios that consider an aquifer that is underlain by intact bedrock (Chapter 6), an aquifer intersected by a highly permeable layer and underlain by intact bedrock (Chapter 7), and an aquifer that is underlain by fractured bedrock (Chapter 8) and running contaminant transport simulations for 60 years to assess the performance of different designs on an existing plume

• Generating a highly heterogeneous aquifer model, calibrating groundwater flow model, and running contaminant transport simulations for cases that consider a heterogeneous aquifer underlain by intact bedrock (Chapter 9) and a heterogeneous aquifer that is underlain by fractured bedrock (Chapter 10)
Chapter 2

Hydraulic Isolation System Design

Designs that are solely passive or a combination of both passive and active technologies are considered and depicted in this chapter. These designs will be demonstrated on conceptual aquifer models in Chapters 3 and 4 and considered for use in later chapters.

2.1 Gravel Trench

The simplest passive hydraulic source zone isolation design is a gravel trench. This can be constructed using an excavator to dig a trench around a source zone. The trench can then be back-filled with gravel or other unconsolidated material (Ankeny and Forbes, 1997). The upper limit hydraulic conductivity of porous media is around 1 cm/s for gravel (Freeze and Cherry, 1979) and this will be the value assigned for the hydraulic conductivity of the trench. The porosity assigned in this design will be 0.45 and the anisotropy of the trench is assigned a value of 1. If the contrast in hydraulic conductivity between the gravel trench and the aquifer is high enough, then a gravel trench may perform as an effective hydraulic isolation design. A depiction of a gravel
trench hydraulic source zone isolation design is provided in Figure 2.1.

![Gravel Trench Hydraulic Barrier](image)

Figure 2.1: A gravel trench hydraulic barrier.

### 2.2 Gravel Trench and Pipe

Passive hydraulic isolation is based on the idea that streamlines can be treated as impervious boundaries. For this to be the case, the trench must accommodate significantly more flow than the surrounding porous media. Darcy’s Law:

\[ Q_{\text{darcy}} = KA_{\text{trench}} \frac{\Delta h}{L} \]  

(2.1)

where \( Q_{\text{darcy}} \) is the volumetric flow rate in porous media, \( K \) the hydraulic conductivity, \( A_{\text{trench}} \) the cross sectional area of the gravel trench, and \( \frac{\Delta h}{L} \) the hydraulic gradient, can be used to describe flow through the gravel trench. Increasing the hydraulic conductivity of the trench can reduce the head gradient across a site if the volumetric flow rate is kept constant. This can be done by using coarser material than the gravel proposed in the previous section. However, the porous media used in the trench will naturally have an upper limit and for highly conductive aquifers, an even greater contrast in hydraulic conductivity between the aquifer and the trench may be needed in order to isolate a contaminant source zone. To allow for a greater amount of flow to be accommodated in the permeable hydraulic barrier, the installation of
pipe should be considered during construction of the trench. The resulting design would be similar to the French drain used in Davis and Stansfield (1984) to divert flow. However, instead of only diverting the flow, the goal is to hydraulically isolate a source zone as much as possible and the inclusion of perforated pipes in a gravel trench that surrounds a contaminant source may do so.

The subsequent increase in flow due to the inclusion of pipes can be estimated using a weighted average approach if laminar and parallel flow are assumed in both the pipe and trench. Incoming flows through the perforations or slots of the pipe are neglected for simplicity. Darcy’s law is used to describe the flow through the gravel in the trench. The Darcy-Weisbach equation (2.2) (Elger et al., 2012):

$$\frac{\Delta h}{L} = \frac{fV^2}{2gD}$$

where $f$ is the friction factor, $V$ the velocity in the pipe, $g$ the gravitational constant, and $D$ the diameter of the pipe, can be used to describe flow through the pipes in this design. The friction factor can be obtained from a Moody Diagram, but if the flow field is fully developed, laminar, and steady, then the friction factor can be calculated using $f = \frac{64}{Re}$. The parameter $Re$ is the Reynolds number, which is calculated as

$$Re = \frac{V \rho D}{\mu},$$

where $V$ is the velocity, $\rho$ the fluid density, $D$ the diameter of the pipe, and $\mu$ the dynamic viscosity. For laminar flows, the head loss is unaffected by the pipe roughness.

If the equations for the friction factor and Reynolds number are substituted into the Darcy-Weisbach equation (2.2), the following equation is obtained:

$$\frac{\Delta h}{L} = \frac{64\mu V}{2\rho g D^2}$$

(2.3)
Since $V = \frac{Q_{pipe}}{A_{pipe}}$, where $Q_{pipe}$ is the volumetric flow rate through the pipe and $A_{pipe}$ the cross sectional area of the pipe, $\frac{Q_{pipe}}{A_{pipe}}$ can be substituted in place of the velocity term. If the cross sectional area of the pipe $D^2 \pi / 4$ is substituted for $A_{pipe}$ then equation (2.3) yields:

$$\frac{\Delta h}{L} = \frac{128 Q_{pipe} \mu}{\rho D^4 g \pi}$$

(2.4)

When equation (2.4) is rearranged to solve for the the volumetric flow rate in the pipe, $Q_{pipe}$, the following form is obtained

$$Q_{pipe} = \frac{D^4 g \pi \Delta h}{128 \gamma L}$$

(2.5)

where $\gamma$ is equal to the kinematic viscosity. By adding the volumetric flow rate through the pipe and porous media together, a total volumetric flow rate can be calculated that would take the form:

$$Q_{total} = \frac{D^4 g \pi \Delta h}{128 \gamma L} + K A_{trench} \frac{\Delta h}{L}$$

(2.6)

where $A_{trench}$ is the cross sectional area of the trench. Instead of accounting for flow through the pipe and the porous material separately, an effective hydraulic conductivity term, $K_{eff}$, that accounts for flow through the porous media and pipe can be used to relate the hydraulic gradient and the cross sectional area of the trench and pipe. $A_{total}$, to the total volumetric flow rate:

$$K_{eff} A_{total} \frac{\Delta h}{L} = \frac{D^4 g \pi \Delta h}{128 \gamma L} + K A_{trench} \frac{\Delta h}{L}$$

(2.7)

Since the hydraulic gradient is the same for the pipe and the porous material,
it can be divided out. Rearranging the equation, a weighted average of flow through
the pipe and porous material is obtained. Equation (2.8) can be used to calculate
the effective hydraulic conductivity of the trench and relate it to different hydraulic
isolation design parameters such as the number of pipes, diameter of the pipes, and
area of the trench. The volumetric flow rate from pipe of the same diameter can be
weighted by \( n \) which represents the number of pipes installed in the trench.

\[
K_{eff} = \frac{n \frac{D^4 g \pi}{128 \gamma} + K A_{trench}}{A_{total}}
\]  

(2.8)

This approach is particularly attractive since it is not necessary to know the
hydraulic gradient of the site to calculate the effective hydraulic conductivity of a
trench design with pipes. An example calculation is worked out in Appendix A and
the result is compared with a COMSOL simulation of flow in a porous medium and
pipe for the same scenario. Equation 2.8 provides a useful estimate that is within a
factor of two for the example considered. A sketch of a design with gravel trench and
pipe is depicted in Figure 2.2.

Figure 2.2: A hydraulic barrier that consists of a gravel trench and pipe.
2.3 Gravel Trench, Pipe, and Pumping and Injection Wells in the Trench

Achieving a contrast in hydraulic conductivity high enough to totally isolate a contaminant source zone may require a significant number of pipes in some environments. The effect of clogging on the perforated pipes and the hydraulic conductivity of the trench cannot be neglected. Geotextiles such as what was used in Davis and Stansfield (1984) and US EPA (2004), help reduce clogging, but do not totally eliminate it from affecting the performance of a passive hydraulic barrier. Installing wells in the trench would be straightforward and monitoring wells will likely be needed in a gravel trench to monitor its performance. Making use of extraction and injection in the trench could work to improve the performance of the source zone isolation design by neutralizing the head in the hydraulic barrier. Figure 2.3 shows a design that includes a gravel trench with pipe and pumping and injection wells in the trench.

Figure 2.3: A hydraulic isolation system that consists of a gravel trench with pipe and injection and extraction wells.
2.4 Gravel Trench, Pipe, and Source Zone Pumping

A design that is completely passive may not work for every hydrogeologic scenario. In some cases, a combination of active and passive designs may provide the most effective design. In Ankeny and Forbes (1997), plants or a pumping well are suggested to be placed in the source zone so that an inward and upward gradient can be created. Such a design may be necessary for hydrogeologic sites where there is a significant amount of uncertainty about the flow field or the geology of the site. The inclusion of a hydraulic barrier created from a gravel trench that includes pipe would reduce the number of wells needed in the source zone and lower the flow rate required to hydraulically isolate the source. A schematic showing a gravel trench with pipe and source zone pumping is displayed in Figure 2.4.

Figure 2.4: A hydraulic isolation system design that includes source zone pumping in addition to a gravel trench with pipe.
Chapter 3

Homogeneous Conceptual Model

Different designs of a hydraulic barrier and design parameters of a gravel trench such as the depth, thickness, hydraulic conductivity, and the inclusion of wells and their effect on the plume mass of a tracer are tested on a conceptual model for an aquifer.

3.1 Flow Conceptual Model

A rectangular, homogeneous, unconfined aquifer at steady state is considered for the conceptual model in this chapter. Constant heads of 15.24 m (50 ft) and 14.94 m (49 ft) are assigned to two of the aquifer boundaries. No flow boundaries are assigned to the remaining faces of the aquifer. The geometry of the aquifer model and boundary conditions are depicted in Figure 3.1.
Figure 3.1: The grid and model boundary conditions for the conceptual model. A constant head of 15.2 m is assigned to the upgradient boundary and 14.9 m to the down gradient boundary. The remaining sides are no flow boundaries.

The top of the grid has an elevation of 15.24 m (50 ft) and the bottom of the model has an elevation of 0 m. The groundwater model has dimensions of 121.92 m (400 ft) × 60.92 m (200 ft) × 15.24 m (50 ft). The grid contains 500,000 cells with uniform dimensions of 0.61 m (2 ft) × 0.61 m (2 ft) × 0.61 m (2 ft). A hydraulic conductivity of 8.53 m/d (28 ft/d) is assigned to the aquifer. The hydraulic conductivity was chosen to model an aquifer matrix that consists of medium sized sand grains (Freeze and Cherry, 1979). A vertical anisotropy of 10 is used to account for heterogeneity due to the layering that commonly occurs in sedimentary systems. The model parameters are summarized in Table 3.1.
Table 3.1: Dimensions and parameters of the conceptual aquifer model.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grid Top</td>
<td>15.2 m</td>
</tr>
<tr>
<td>Grid Bottom</td>
<td>0 m</td>
</tr>
<tr>
<td>Model Dimensions</td>
<td>121.9 m × 60.9 m × 15.2</td>
</tr>
<tr>
<td>Cell Dimensions</td>
<td>0.61 m × 0.61 m × 0.61 m</td>
</tr>
<tr>
<td>Number of Cells</td>
<td>500,000</td>
</tr>
<tr>
<td>Hydraulic Conductivity of Aquifer</td>
<td>28 ft/d</td>
</tr>
<tr>
<td>Vertical Anisotropy</td>
<td>10</td>
</tr>
</tbody>
</table>

3.2 Groundwater Model

The groundwater simulation result of the conceptual aquifer model described in the previous section, with no hydraulic barrier, is summarized in Figure 3.2. Figure 3.2a, a perspective view of the flow field, displays groundwater head contours that are evenly distributed. MODPATH (Pollock, 2016) pathlines are shown in Figure 3.2b, in plan view, displaying a one dimensional flow field that only has a horizontal component. In Figure 3.2c, flow vectors uniform in magnitude and with the same horizontal orientation are displayed along a cross section through the aquifer model. The travel time across the aquifer for this base case is about 4.5 years.
Figure 3.2: Groundwater simulation results for the conceptual model under natural conditions.

3.3 Hydraulic Barrier Inclusion

The hydraulic barrier design to be included in this conceptual model is a rectangular trench that has been back-filled with gravel (see Figure 2.1). The gravel trench included in the conceptual aquifer model has a length of 45.1 m (148 ft), a width of 25.6 m (84 ft), a thickness of 0.61 m (2 ft), and a perimeter of 141 m (464 ft). It is installed to a depth of 15.24 m (50 ft) so that it fully penetrates the aquifer. A hydraulic conductivity of 853 m/d (2800 ft/d) is assigned to the gravel trench. The design parameters are summarized in Table 3.2 and the inclusion of the gravel trench in the model is shown in Figure 3.3.
Table 3.2: Dimensions and properties of the gravel trench design.

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Length</strong></td>
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</tr>
<tr>
<td><strong>Width</strong></td>
<td>25.6 m (84 ft)</td>
</tr>
<tr>
<td><strong>Thickness</strong></td>
<td>0.61 m (2 ft)</td>
</tr>
<tr>
<td><strong>Depth</strong></td>
<td>15.2 m (50 ft)</td>
</tr>
<tr>
<td><strong>Perimeter</strong></td>
<td>141 m (464 ft)</td>
</tr>
<tr>
<td><strong>Hydraulic Conductivity of Trench</strong></td>
<td>853 m/d (2800 ft/d)</td>
</tr>
</tbody>
</table>

Figure 3.3: Hydraulic barrier inclusion in conceptual model simulations. The units for the horizontal hydraulic conductivity scale are in m/d.

3.4 Hydraulic Barrier Simulation

Under natural conditions the flow field is uniform and one dimensional, but when a gravel trench is installed, the flow field is significantly altered (Figure 3.4). A head equalization effect takes place across the gravel trench. Along the upgradient dimension the head is reduced and along the downgradient dimension the head is raised. If the head gradient through the center of the gravel trench is calculated for this design and compared with the base case head gradient, a decrease from 0.0025 to 0.0011 is observed (a reduction of 56%). The MODPATH pathlines in Figure 3.4b show that the travel time across the isolated zone alone is 4 years with the inclusion of the permeable barrier whereas under natural conditions the travel time across the
entire model was 4.5 years. Most of the upstream groundwater in the capture zone of the gravel trench preferentially flows within it and exits downgradient despite some flow through the protected zone (Figures 3.4b and 3.4c). The reduction in the velocity through the source zone is evident with the decrease in magnitude of the flow vectors in the isolated zone which is a result of the reduction in head gradient (Figure 3.4c).

Figure 3.4: Groundwater flow simulation results for the conceptual model upon inclusion of a gravel trench hydraulic barrier (black).

3.5 Transport Conceptual Model

The results from the previous section demonstrate how the gravel trench modifies the hydraulic head in order to achieve some degree of source zone containment using groundwater simulation results. This approach quantifies the performance of a hydraulic barrier by calculating the reduction in head gradient and the travel time across the source zone. The next step in assessing the performance of a hydraulic
source zone isolation design is to create a source zone and perform contaminant transport simulations with MT3DMS. To do so, a rectangular source zone with dimensions of 6.1 m (20 ft) x 25.6 m (42 ft) x 3.05 m (10 ft) is created inside the rectangular hydraulic barrier. A constant concentration of 100 mg/L was assigned to the source in layers 10 to 14 (6.1 m to 9.14 m below the top of the grid) of the model.

![Tracer](a) Top view ![Tracer](b) Side view (Z magnification=3)

Figure 3.5: The location of the specified concentration tracer used to represent a source zone in relation to the hydraulic barrier (red).

The aquifer porosity remains unchanged at 0.3 and a longitudinal dispersivity of 0.03048 m (0.1 ft) assigned to the aquifer. A low longitudinal dispersivity is used to minimize the effects of dispersion since advection is the primary mechanism of transport. Emphasis is placed on the hydraulics of source zone isolation. The ratio of horizontal transverse dispersivity to longitudinal dispersivity (TRPT) and the ratio of vertical transverse dispersivity to longitudinal dispersivity (TRVT) were set to 0.1 and 0.01 respectively. The tracer was given a half life of 1 year to account for biodegradation of the plume and the contaminant transport simulations were run for 30 years to represent the lifetime of a remediation project. The transport conceptual model is summarized in Table 3.3 and depicted in Figure 3.5. The contaminant transport boundaries of the model are open at the inlet and outlet for the model and closed along the remaining boundaries. The central difference method was selected to solve the advective term in the transport equation.
Table 3.3: Summary of the properties for the transport simulation

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Source Concentration (Constant)</td>
<td>100 mg/L</td>
</tr>
<tr>
<td>Source Zone Dimensions</td>
<td>6.1 m × 25.6 m × 3.05 m</td>
</tr>
<tr>
<td>Porosity</td>
<td>0.3</td>
</tr>
<tr>
<td>Longitudinal Dispersivity</td>
<td>0.03048 m</td>
</tr>
<tr>
<td>TRPT</td>
<td>0.1</td>
</tr>
<tr>
<td>TRVT</td>
<td>0.01</td>
</tr>
<tr>
<td>Tracer’s Half-life</td>
<td>1 year</td>
</tr>
<tr>
<td>Time</td>
<td>30 years</td>
</tr>
</tbody>
</table>

Plume mass is used throughout this study as a quantitative measure of hydraulic source zone isolation instead of other metrics such as mass discharge. Plume mass is a simple metric to calculate while providing insight into the performance of a hydraulic isolation design. If a source zone and plume are defined, then equation (3.1) would suffice for a plume mass calculation that is representative of the scenario being considered. Since a constant concentration source zone is used in this study, under the natural gradient, the plume mass will increase over time until a steady state is reached. If a hydraulic source zone isolation design is installed and a preexisting plume used as initial conditions for a transport simulation, then the plume mass will decrease until a new, smaller steady state plume mass is obtained. Throughout the rest of this work, the plume will be defined as all the cells outside of the hydraulic barrier and the source zone consisting of the cells inside the hydraulic barrier. The mass in each of the selected cells are calculated using the following equation:

\[ M = Vol \cdot \phi \cdot Concentration, \]  

where \( M \) is mass, \( Vol \) the volume of the cell, \( \phi \) the porosity, and \( Concentration \) the concentration value in the cell. The mass is computed in each cell and then summed to get a total plume mass.
Figure 3.6: A depiction of how the source zone and the plume are differentiated for plume mass calculations. Only cells outside of the source zone will be used to calculate plume mass.

### 3.6 Sensitivity Analysis

#### 3.6.1 Trench Effective Hydraulic Conductivity

To gain insight into the effects of the hydraulic conductivity of a hydraulic barrier design on plume mass, the hydraulic conductivity of the trench was simulated for values of 853 m/d (2800 ft/d), 8,530 m/d (28,000 ft/d), and 85,300 m/d (280,000 ft/d). Although the values selected for the sensitivity analysis are not typically seen in porous media, the insertion of perforated or slotted horizontal pipes within a gravel trench can result in effective hydraulic conductivity values that are unnaturally high using equation (2.8).

For example, using equation (2.8), an effective hydraulic conductivity of 8,530 m/d can be estimated, for the gravel trench with the properties and dimensions described in Table 3.2, if six pipes each with a diameter of 5.08 cm (2 in) are installed in the trench. The head gradient along one of the side dimensions of the trench with an effective hydraulic conductivity of 8,530 m/d can be calculated to be $5.34 \times 10^{-4}$ if the conceptual model described in section 3.1 is used. By applying Darcy's law, the volumetric flow rate into a trench with an effective hydraulic conductivity of 8,530 m/d can be estimated.
m/d and cross sectional dimensions of 0.61 m by 15.2 m can be calculated to be 42.9 m$^3$/d. The velocity in each of the pipes can be estimated by dividing the 42.9 m$^3$/d by the total cross sectional area of pipe installed in the trench. This will yield a velocity of about 0.04 m/s through each pipe.

Increasing the effective hydraulic conductivity of a trench that fully penetrates an aquifer results in a greater degree of source zone isolation. Figures 3.7 and 3.8 show the hydraulic isolation transport simulation results for hydraulic barrier designs with different hydraulic conductivities in plan view (Figure 3.7) and in cross sectional view (Figure 3.8) after 30 years of simulation. The initial conditions are shown in Figures 3.7a and 3.8a have reached a steady state and persist due to the constant concentration source. Figures 3.7b and 3.8b display the results for the scenario where a gravel trench is installed as a hydraulic barrier. The reduction in plume mass upon inclusion of a gravel trench is significant in comparison to the initial conditions. If the gravel trench design is modified and pipes included such that the effective hydraulic conductivity of the hydraulic barrier is 8,530 m/d, then there is an additional reduction in plume mass shown in Figure 3.7c and 3.8c. For a design that includes pipe in the gravel trench such that the hydraulic conductivity becomes 85,300 m/d, Figures 3.7d and 3.8d display a source zone is essentially completely isolated.
Figure 3.7: Tracer concentrations after 30 years for different effective hydraulic conductivity values. The results show a k=12 plan view. The units for the scale are mg/L.
Figure 3.8: Tracer concentrations after 30 years for different trench hydraulic conductivity values. The results are shown for the cross section i=50. The units for the scale are mg/L. (Z magnification = 3)
The response to the hydraulic barrier is initially transient and begins with the flushing out of the preexisting plume. Since biodegradation is taking place, lower concentrations are closest to the boundary of the model and are the first to reach the model boundary and plume mass is reduced. After the lower concentrations of the preexisting plume have exited the model, the higher concentrations that were farther from the boundary begin to get flushed out and plume mass gets reduced at a faster rate. This is followed by a pseudo-steady state after 1.5 years, where the system begins to stabilize until a steady state is reached. Figure 3.9 quantifies the effect that the different designs have on source zone isolation with a semi-log plot of plume mass for the different designs considered. The plume mass results are normalized by the initial plume mass (Figures 3.7a and 3.8a). Figure 3.9 shows a decrease in plume mass of 74% to 26% of the initial plume mass after 30 years of simulation for a gravel trench (K=853 m/d). Despite the reduction in plume mass for a gravel trench design, there is still room for improvement. Increasing the effective hydraulic conductivity of the trench to 8,530 m/d resulted in an even further reduction of 99.7% to 0.003 of the initial plume mass. The plume mass computed for a hydraulic barrier design with an effective hydraulic conductivity of 85,300 m/d is nearly $5.3 \times 10^{-6}$ the initial plume mass or a 99.9994% reduction, which confirms the isolation of the source zone.
3.6.2 Trench Depth

The depth of the hydraulic barrier is another key parameter to consider. The previous scenarios examined cases where the trench fully penetrated the aquifer. Realistically, it may be difficult to ensure that a hydraulic barrier fully penetrates an aquifer to a low permeability confining unit or bedrock. This difficulty arises from the heterogeneity of a site, uncertainties or discontinuities in the aquifer/confining unit contact, or perhaps fractured bedrock underlying the aquifer of interest, which would have an equivalent effect. The depth of the hydraulic barrier was modified and simulated for scenarios where depth was set to 15.2 m (50 ft), 14.6 m (48 ft), and 12.2 m (40 ft). The hydraulic conductivity of the hydraulic barrier is kept constant at 853 m/d, consistent with the design of a gravel trench. The thickness of the gravel trench was also kept constant at 0.61 m (2 ft).

Decreasing the depth of the gravel trench results in a poorer hydraulic isolation design. The source zone is mobilized from a vertical component of flow that bypasses the hydraulic barrier and contributes to tracer concentrations outside of the hydraulic

![Normalized plume mass as a function of time for different trench effective hydraulic conductivity values. The hydraulic conductivity of the aquifer is 8.53 m/d.](image-url)
barrier. Planar views of the effect of different design depths for contaminant transport simulations for a gravel trench are shown in Figures 3.10. The results are shown after 30 years of simulation and depict a larger plume mass outside of the hydraulic barrier for partially penetrating designs of 14.6 m and 12.2 m.

The cross sectional view in Figure 3.11 provides even further insight into the affect of depth. It becomes clear partially penetrating designs can limit the success of a hydraulic isolation design. Upon decreasing the depth of the gravel trench, a vertical component of flow begins to develop and the plume begins to spread to greater depths. In Figures 3.11a and 3.11b, there does not appear to be any advective transport of the tracer in the vertical direction, only some slight dispersion of the tracer. In contrast, Figure 3.11c, where the gravel trench is nearly installed to the bottom of the aquifer (0.61 m above the confining unit), there is some vertical advective transport taking place in the source zone that is not present in Figure 3.11b, the fully penetrating design. Figure 3.11d displays the results for the gravel trench design that is nearly 3 m above the bottom of the aquifer. The vertical advective transport of the tracer is amplified even further in comparison to Figure 3.11c and the vertical extension of the plume is enhanced. Partially penetrating hydraulic barrier designs can allow the tracer a means to bypass the hydraulic isolation design. This effect can not only reduce the effectiveness of source zone isolation, but it can also make clean up efforts more challenging. Enhanced vertical transport of a contaminate could potentially result in an increase in the volume of the aquifer in contact with a contaminant or lead to the contamination of a clean, underlying aquifer.
Figure 3.10: Tracer concentrations after 30 years for different gravel trench (K=853 m/d) depth designs. The results show a k=12 plan view. The units for the scale are mg/L.
Figure 3.11: Tracer concentrations after 30 years for different gravel trench (K=853 m/d) depth designs. The results are shown for the cross section i=50. The units for the scale are mg/L. (Z magnification = 3)
Design performance is significantly impacted by the distance between a low permeability unit and the bottom of the trench. The response to perturbation of the system for the partially penetrating designs considered in this chapter begins with a slight increase in plume mass due to the relatively high concentration vertical contributions to the plume from the source zone which peaks at 60 days. This short-lived increase is followed by a decrease in plume mass due to the reduction in the head gradient across the source zone and an increase in the mass exiting the model. In contrast to Figure 3.9, the reduction in plume mass in Figure 3.12 is not immediately followed by a steady state. Rather, there is a rebound in plume mass that begins after 1.3 years for the 12.2 m design and 1.5 years for the 14.6 m design. The rebound effect is due to the vertical flow that partially penetrating designs induce. Figure 3.12 shows that as the depth of the gravel trench is reduced, the rebound effect is enhanced. For a design that is installed to a depth of 14.6 m, the plume mass is 71% of the initial plume mass or experiences a reduction of 29%. A design that considers a gravel trench installed to a depth of 12.2 m results in a plume mass that is 84% of the initial plume mass (16% reduction). The fully penetrating gravel trench, which is installed to a depth of 15.2 m outperforms both of these designs with a plume mass that is 26% of the initial plume (74% reduction). If there is a higher degree of anisotropy in the aquifer, then the difference between the performance of the two designs would likely be less because the vertical transport of the tracer would be reduced for partially penetrating designs.
3.6.3 Trench Thickness

The thickness of the trench is another parameter that will affect the mass discharged from the source zone into the plume. This section compares the results of contaminant transport simulations after 30 years for different gravel trench designs that have thickness values of 0.61 m (2 ft), 1.22 m (4 ft), and 2.44 m (8 ft) respectively. The hydraulic conductivity of the gravel trench is 853 m/d, and because there are no pipes in it, the effective hydraulic conductivity does not change with thickness which would be the case if pipe is included and equation (2.8) used. The gravel trench hydraulic barrier designs considered in this section each reach a depth of 15.2 m and fully penetrate the aquifer. It is important to bear in mind that limitations of cost and equipment may make it difficult to construct trenches that are very thick, thus, using a higher effective trench conductivity by installing pipe may be a preferred alternative to increasing the trench thickness. Nevertheless, trench thickness is still an important design parameter to consider.
A gravel trench design that has a thickness of 1.22 m achieves a greater degree of source zone isolation and has less mass discharging out of the source zone than a 0.61 m thick trench (Figures 3.13c and 3.14c). If the thickness of the gravel trench is increased even further to 2.44 m, then the mass discharge is reduced even further due to the greater degree of source zone isolation.

The plume mass plot in Figure 3.15 allows for comparison between the gravel trench designs with different thicknesses. A gravel trench with a thickness of 0.61 m results in a steady state plume mass that reduces to 26% the initial plume mass (74% reduction). Increasing the thickness to 1.22 m results in a steady state plume mass that reduces to 12% of the initial plume mass (88% reduction). An additional increase in gravel trench thickness to 2.44 m improves the performance of the gravel trench even further and has a steady state plume mass that reduces to 4.4% of the initial plume mass (95.6% reduction). Additionally, as trench thickness increases, the time to reach steady state also increases slightly.
Figure 3.13: Tracer concentrations after 30 years for different gravel trench (K=853 m/d) thickness designs. The results show a k=12 plan view. The units for the scale are mg/L.
Figure 3.14: Tracer concentrations after 30 years for different gravel trench (K=853 m/d) thickness designs. The results are shown for the cross section i=50. The units for the scale are mg/L. (Z magnification = 3)
3.7 Hydraulic Isolation Designs

3.7.1 Gravel Trench with Pumping and Injection Wells in the Trench

The performance of a hydraulic barrier design that consists of a gravel trench (K=853 m/d) and low flow rate pumping and injection wells screened in the trench was also examined. Three pumping and injection wells are included and their locations in the gravel trench are shown in Figure 3.16. Each extraction well pumps at a rate of -5.4 L/min (-1.43 gpm) and each of the injection wells inject at a rate of 5.4 L/m (1.43 gpm). Setting the pumping and injection rates at equal magnitudes allows for a net water balance of zero.
Figure 3.16: Top view of the location of pumping and injection wells (yellow) in relation to the gravel trench hydraulic barrier. The magnitude of the volumetric flow rate (Q) for each well is 5.4 L/min.

The impact that the inclusion of pumping and injection have on source zone isolation is immediate. Figures 3.17 and 3.18 show that after only 60 days, the plume is nearly cut off from the source zone. After 360 days, the plume is detached in some areas and even more so after 30 years. Only concentrations less than 0.01 mg/L are present outside of the isolated zone.
Figure 3.17: Plume concentrations at selected times after inclusion of a gravel trench with pumping and injection wells. The results shown are for a plan view of $k=12$. The units for the scale are mg/L.
Figure 3.18: Plume concentrations at selected times after inclusion of a gravel trench with pumping and injection wells. The results are for the cross section i=50. The units for the scale are mg/L. (Z magnification=3)
The head gradient between the upstream and downstream dimensions of the hydraulic barrier is calculated for comparison with the base case and the gravel trench designs. The head gradient for the base case scenario is 0.0025 and with a gravel trench as a hydraulic barrier the head gradient is reduced to 0.0011. The inclusion of pumping and injection wells in the gravel trench results in a decrease of 2 orders of magnitude from 0.0011 to 0.000016 in hydraulic gradient in comparison to the design that consists of only a gravel trench.

The implications this has on hydraulic source zone isolation are depicted in Figures 3.19 and 3.20. If a contaminant with low MCL is considered and the scale used for concentration is a log scale, then the difference between the scenario where no hydraulic barrier is used and a gravel trench design is used is not as pronounced as what was depicted in Figures 3.7 and 3.8. In this case, the use of pumping and injection wells may be needed to reduce the concentrations in the plume even further. There is a four order of magnitude reduction in the concentrations present in the gravel trench with pumping and injection wells design in comparison to the gravel trench design plume concentrations.
Figure 3.19: Contaminant transport simulation results for different hydraulic barrier designs after 30 years of simulation. The results are in plan view for k=12. The units for the scale are mg/L.
Figure 3.20: Contaminant transport simulation results for different hydraulic barrier designs after 30 years of simulation. The results are for the cross section i=50. The units for the scale are mg/L. (Z magnification=3)
The introduction of pumping and injection wells improves the performance of the hydraulic isolation design significantly in comparison to that of the gravel trench design. The normalized plume mass is reduced by four orders of magnitude in comparison to the gravel trench design (Figure 3.21). The response of the plume mass to the inclusion of the gravel trench with wells design is very similar to the K=85,300 m/d case shown in Figure 3.9. The initial response time is transient until about 1.5 years. From 1.5 years to 10 years, the plume mass is in a pseudo-steady state until it reaches a steady state at $2.8 \times 10^{-5}$ the initial plume mass (99.9972% reduction). This confirms that one plausible alternative or addition to the inclusion of pipe in a gravel trench is wells screened in the trench, which as demonstrated in Figure 3.21, can also be an effective means to achieve source zone isolation.

![Figure 3.21: Normalized plume mass as a function of time comparing a gravel trench design with a gravel trench with wells design.](image)

For a scenario where the gravel trench and wells do not fully penetrate the aquifer, the vertical transport effect seen in Figure 3.11 will occur and may reduce the effectiveness of the system in reaching remediation goals. Figure 3.22 shows the simulation results for a scenario where pumping and injection wells and a gravel
trench (K=853 m/d) are included in the hydraulic barrier design, but only penetrate the aquifer to a depth of 12.2 m, leaving 3 m between the hydraulic barrier and the bottom of the aquifer. The vertical transport effect takes place in the source zone and bypasses the hydraulic barrier. The concentrations that bypass the hydraulic barrier are relatively small for this case and this design could be successful depending on the goals of the project. Although, it is important to note that if the half-life of the tracer were longer than one year or if there was a lower degree of anisotropy in the model, the concentrations that bypass the hydraulic barrier would be higher than what has been demonstrated for this case.

![Figure 3.22: Plume concentrations after 30 years of simulation for a gravel trench with pipe and pumping and injection wells that penetrate an aquifer to a depth of 12.2 m. The units for concentration are in mg/L.](image1)

(a) Planar cross section k=12

(b) i=50 cross section (Z magnification=3)

Figure 3.22: Plume concentrations after 30 years of simulation for a gravel trench with pipe and pumping and injection wells that penetrate an aquifer to a depth of 12.2 m. The units for concentration are in mg/L.

The normalized plume mass plotted as a function of time for the cases considered confirms that despite the vertical transport effect, the normalized plume mass reduced to 0.00095 the initial plume mass (a reduction of 99%). Figure 3.23 shows that in comparison to the fully penetrating design of a gravel trench with wells, there
is a 3 order of magnitude increase in plume mass. Nevertheless, the partially pen-etrating design does contribute to source zone isolation efforts and may still perform successfully depending on the chemical properties of the contaminant, heterogeneity in the aquifer, and the goals of the remediation project.

Figure 3.23: Normalized plume mass as a function of time comparing a fully pen-etrating gravel trench, fully penetrating gravel trench with wells design, and a partially penetrating gravel trench with wells design.
Chapter 4

Heterogeneous Conceptual Model

The results of the homogeneous conceptual model simulations in Chapter 3 demonstrate the validity of the concept of using a passive hydraulic barrier for hydraulic source zone isolation and the performance of various designs. However, homogeneity is a simplification of reality. In this chapter, a random, heterogeneous conceptual model was generated using T-Progs and the sensitivity of design parameters such as effective hydraulic conductivity, depth, and thickness on performance analyzed.

4.1 TPROGS

T-Progs (Carle, 1999) is a geostatistical software package that uses Markov chains to stochastically simulate spatial variability. Markov chains are processes that are based on the notion that the future is only dependent on the present and not the past. Markov chains can also be applied to describe spatial processes. When this is the case, Markov Chains can be used in a one dimensional spatial application where spatial occurrences are conditioned to the nearest data (Carle, 1999).
Transition probabilities define the likelihood that a Markov Chain moves from one state to another. Figure 4.1 provides a schematic of how transition probabilities are used in a spatial context in T-Progs. Essentially, if facies $j$ is present at location $\vec{x}$, the likelihood that a different facies, $k$ occurs at location $\vec{x} + \vec{h}$ is the transition probability, $t_{j,k}$. The equation used to describe this relation is:

$$t_{j,k}(\vec{h}) = \Pr(k \text{ occurs at } \vec{x} + \vec{h} | j \text{ occurs at } \vec{x})$$ (4.1)

Figure 4.1: Schematic representation of a transition probability. Adapted from Carle (1999).

Three characteristics are used to relate the transition probability to the lag distance: transition rate, mean length, and mean proportions. These properties are depicted graphically in Figure 4.2. The transition rate is used to define the slope of the transition probability curve as the lag distance approaches zero. Mean length corresponds to the mean lens length of a specified material while mean proportions represents the volume fraction of a material in the area being modeled.
The parameters needed to define a Markov Chain can be measured automatically from borehole information or specified. To generate the heterogeneous conceptual model in this chapter, the mean proportions and lens lengths are specified to create the Markov Chains. Two materials are considered for this demonstration, sand and fine sand. Fine sand is set as the background material. The mean proportion for sand is 0.33 and the mean proportion for fine sand is 0.67. The sand lens lengths in the Z direction is 0.61 m (2 ft) and the fine sands lens length is 2.74 m (9 ft). Lens ratios are then specified using the following equation:

\[ \text{Lens Ratio} = \frac{L_{XorY}}{L_z} \quad (4.2) \]

where \( L_{XorY} \) represents the lens length in the X or Y direction (laterally) and \( L_z \) the lens length in the Z direction (vertically). The lens ratio in the X direction, which is the dominant direction of flow is set as 18 for the sand and 8.12 for the fine sand. The lens ratio in the Y direction is set as 8 for the sand and 3.61 for the fine sand. These
values were selected using a trial and error approach until a heterogeneous spatial
distribution that displayed a sufficient degree of lateral continuity in the model was
obtained. The T-Progs parameters used to generate the Markov Chains in this chapter
are summarized in Table 4.1. An example of 1D Markov Chains in the Z direction
computed using the parameters in Table 4.1 is shown in Figure 4.3.

Table 4.1: Parameters used to define the Markov chains in T-Progs

<table>
<thead>
<tr>
<th>Material</th>
<th>Proportion</th>
<th>Lens Length (Z)</th>
<th>Ratio (X)</th>
<th>Ratio (Y)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>0.33</td>
<td>0.61 m</td>
<td>18</td>
<td>8</td>
</tr>
<tr>
<td>Fine Sand</td>
<td>0.67</td>
<td>2.74 m</td>
<td>8.12</td>
<td>3.61</td>
</tr>
</tbody>
</table>

Figure 4.3: Markov Chains in the Z-direction using the parameters specified in Table 4.1.

The grid used for the T-Progs realization generated in this chapter is the
same as the grid used for the homogeneous grid. The attributes of the parameters
summarized in Table 4.1 are evident in the T-Progs generated material distribution in Figure 4.4. The realization in Figure 4.4 has sand lenses that are elongated in the X direction as opposed to the Y and Z directions due to the X ratio that was assigned to it. The heterogeneous aquifer model is also dominated by the fine sand material due to the larger mean proportion of 0.67 assigned to it. The fine sand makes up most of the aquifer matrix while the sand lenses have enough continuity to create preferential flow paths and complicate the flow field.

![Figure 4.4](image)

Figure 4.4: Heterogeneous spatial distribution for a conceptual model with two materials. (Z magnification=3)

4.2 Groundwater Model

The groundwater boundaries are the same as the homogeneous case (see Table 3.1). Two materials are considered, a clean sand with hydraulic conductivity of 8.53 m/d (28 ft/d) and a fine sand with a hydraulic conductivity of 0.853 m/d (0.28 ft/d). The vertical anisotropy assigned to both materials is 3. The sand is assigned a
porosity of 0.3 and the fine sand a porosity of 0.2. These properties are summarized in Table 4.2.

Table 4.2: Relevant flow parameters for the T-Progs generated material distribution.

<table>
<thead>
<tr>
<th>Material</th>
<th>Hydraulic Conductivity</th>
<th>Vertical Anisotropy</th>
<th>Porosity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>8.53 m/d</td>
<td>3</td>
<td>0.3</td>
</tr>
<tr>
<td>Fine Sand</td>
<td>0.853 m/d</td>
<td>3</td>
<td>0.2</td>
</tr>
</tbody>
</table>

The head contours are much more complicated when heterogeneity is present in the conceptual model. The head contours are no longer linear and flow is no longer one dimensional. Instead, streamlines are now three dimensional. MODPATH particles are assigned to certain cells to get a representative idea of the flow field and travel time in the aquifer. The refraction of the flow paths and changes in travel time are evident in Figure 4.5.
Figure 4.5: Groundwater simulation results for the heterogeneous conceptual model under natural conditions.

4.3 Hydraulic Barrier Inclusion

The hydraulic barrier was included by creating another material as "trench". This material has the same dimensions as what was described in the homogeneous conceptual model scenario, which is summarized below in Table 4.3. An anisotropy of 1 is assigned to the trench material and its hydraulic conductivity is modified as necessary. The location of the hydraulic barrier and how it is represented in the model is shown in Figure 4.6.
Table 4.3: Dimensions of the hydraulic barrier designs considered in this chapter.

<table>
<thead>
<tr>
<th>Dimension</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
<td>45.1 m</td>
</tr>
<tr>
<td>Width</td>
<td>25.6 m</td>
</tr>
<tr>
<td>Thickness</td>
<td>0.61 m</td>
</tr>
<tr>
<td>Depth</td>
<td>15.2 m</td>
</tr>
<tr>
<td>Perimeter</td>
<td>141 m</td>
</tr>
</tbody>
</table>

(a) XY view   (b) XZ cross sectional view

Figure 4.6: Hydraulic barrier inclusion by adding a new material and the location of the specified tracer concentration.

4.4 Contaminant Transport Model

The contaminant transport model is the same as the homogeneous conceptual model case as well to make comparing the two scenarios easier. A description of the contaminant transport model is given in Table 4.4. Plume mass is used as a metric for source zone isolation and the region defined as the plume is shown in Figure 4.7.
Table 4.4: Summary of the properties of the transport simulation

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Constant Source Concentration</td>
<td>100 mg/L</td>
</tr>
<tr>
<td>Tracer Volume</td>
<td>$6.1 \text{ m} \times 25.6 \text{ m} \times 3.05 \text{ m}$</td>
</tr>
<tr>
<td>Porosity</td>
<td>0.3 (sand) 0.2 (fine sand)</td>
</tr>
<tr>
<td>Longitudinal Dispersivity</td>
<td>0.03048 m</td>
</tr>
<tr>
<td>TRPT</td>
<td>0.1</td>
</tr>
<tr>
<td>TRVT</td>
<td>0.01</td>
</tr>
<tr>
<td>Half-life</td>
<td>1 year</td>
</tr>
<tr>
<td>Time</td>
<td>30 years</td>
</tr>
</tbody>
</table>

Figure 4.7: The selected area (all cells outside of the source zone) defined as the plume and used to calculate the plume mass is highlighted. The isolated area within the hydraulic barrier is considered the source zone and not included in plume mass calculations. Equation 3.1 is used to calculate plume mass.

The base case plume after 30 years of simulation (Figure 4.8) for this heterogeneous model was used as the starting concentration for each of the models run in this chapter. The complexity of transport through the heterogeneous conceptual model is noteworthy, especially in comparison to the base case plume for the homogeneous conceptual model. The plume concentrations are lower than a homogeneous scenario in this case due to the effect of biodegradation. Although the half life in this scenario is also 1 year, impediments to flow due to the abundance of fine sand and the discon-
tinuities in the sand for this conceptual model allows for the plume to biodegrade, or decay, before much of it can be transported to the models downgradient boundary. The plume mass plots used in this chapter are all normalized by the steady state plume mass for the base case scenario plotted in Figure 4.9. The plume begins to interact with the model boundary at 2.5 years and a steady state is reached at about 10 years.

Figure 4.8: The base case plume concentrations after 30 years for the heterogeneous conceptual model.
4.5 Sensitivity Analysis

A sensitivity analysis is performed on key trench parameters in this section. These parameters include trench effective hydraulic conductivity, depth, and thickness. Plume mass is calculated for each case and used as a metric to quantify the performance of different hydraulic barriers. The plume mass calculations are normalized by the steady state base case plume mass and compared. Each contaminant transport simulation is run for 30 years.
4.5.1 Trench Effective Hydraulic Conductivity

The effective hydraulic conductivity of the trench is one of the most important parameters in the design of the permeable hydraulic barrier. Effective hydraulic conductivities of 853 m/d (2,800 ft/d), 8,530 m/d (28,000 ft/d), and 85,300 m/d (280,000 ft/d) are assigned to the hydraulic source zone isolation designs considered in this section. Upon inclusion of the gravel trench design where the hydraulic conductivity of the trench is 853 m/d (Figure 4.10b and 4.11b), plume mass undergoes a significant reduction in comparison to the base case scenario (Figure 4.10a and 4.11a). The plume mass is reduced even further if an effective hydraulic conductivity of 8,530 m/d is used for the gravel trench with pipe design (Figures 4.10c and 4.11c), where the concentrations outside of the hydraulic barrier have nearly completely degraded. Again, if the effective hydraulic conductivity of the hydraulic isolation design is increased by a factor of ten from 8,530 m/d to 85,300 m/d (by increasing the number of pipes used and/or the diameter of the pipes), then the source zone is essentially isolated and natural attenuation of the plume nearly results in the elimination of plume mass outside of the hydraulic barrier (Figure 10d and 11d).
Figure 4.10: Tracer concentrations for different values of trench hydraulic conductivity after 30 years of simulation. The results are for the k=12 plan view. The units for the scale are mg/L.
Figure 4.11: Tracer concentrations for different values of trench hydraulic conductivity after 30 years of simulation. The cross sections are for i=50. The units for the scale are mg/L. (Z magnification =3)
Calculations of plume mass, where the plume is defined as being the region outside of the hydraulic barrier, quantify the performance of each of the hydraulic isolation designs and are summarized in Figure 4.12. Installing a gravel trench for the hydraulic isolation design resulted in a decrease to 0.003 of the initial plume mass (99.7% reduction). An increase in effective hydraulic conductivity from 853 m/d (gravel trench) to 8,530 m/d (gravel trench with pipe) in a greater degree of source zone isolation and a reduction in plume mass to $1 \times 10^{-5}$ of the initial plume mass (99.999% reduction). For a hydraulic isolation design with an effective hydraulic conductivity of 85,300 m/d, the plume mass was essentially reduced to 0 due to complete isolation of the source zone and natural attenuation of the plume. The aquifer response time to perturbation increases as a result of the heterogeneity in the model. The gravel trench design (K=853 m/d) reaches a steady state at about 10 years and the K=8,530 m/d and K=85,300 m/d reach steady state at about 12 years.

Figure 4.12: Normalized plume mass as a function of time for a heterogeneous conceptual model for different hydraulic barrier hydraulic conductivity values.
4.5.2 Trench Depth

In the homogeneous conceptual model, the increase in vertical transport of the plume as a result of decreasing the depth of installation of the passive hydraulic barrier was demonstrated. Stream lines were flowing through the targeted isolated area, mobilizing some of the source, and transporting contaminants in a way that bypassed the passive hydraulic barrier. The same scenario is considered, although, in the presence of the heterogeneity being considered this chapter. A gravel trench, with a hydraulic conductivity of 853 m/d and thickness of 0.61 m, is used for each scenario and the depth of the trench is reduced to 15.2 m (50 ft), 14.6 m (48 ft), and 12.2 m (40 ft) in this section.

Figure 4.13c and 4.14c show the results, after 30 years of simulation, for design that has gravel trench with a depth of 14.6 m. The change in plume geometry is not noticeable in the planar view (4.13c) when compared to a fully penetrating design (4.13b). There appears to be practically no difference in the planar extent of the plume for the hydraulic barrier designs considered in Figures 4.13b, 4.13c, and 4.13d. However, a comparison of the cross sectional views of the plume (Figures 4.14b and 4.14c) does show a subtle change in plume geometry and a slight increase in vertical transport of the plume. Nevertheless, the results are similar the scenario where the passive hydraulic barrier fully penetrates the aquifer to a depth of 15.2 m (Figures 4.14b). An additional reduction in the depth of the trench leads to a slightly more noticeable distortion in plume geometry in Figure 4.14d, but the change is minimal.
Figure 4.13: Tracer concentrations for different values of trench depth after 30 years of simulation. The results are for the k=12 plan view. The unit for concentration is mg/L.
Figure 4.14: Tracer concentrations for different values of trench depth after 30 years of simulation. The cross sections are for i=50. The unit for concentration is mg/L. (Z magnification =3)
Each design reaches steady state at about 10 years. The rebound effect that was observed in Figure 3.12 is not present for the case considered here which may be a result of the tracer’s half-life of one year. If the half-life were longer, perhaps the rebound in plume mass would occur. Figure 4.15 summarizes the normalized plume mass calculations for each of the designs considered in this chapter. The plume mass is reduced from the base case starting concentrations to 0.0084 of the initial plume mass (99.16% reduction), which as shown in Figure 4.14, is hardly different from the 0.009 (99.10% reduction) for a gravel trench constructed to a depth of 14.6 m. An additional reduction in the depth of the gravel trench design to 12.2 m, results in a plume mass reduced to 0.0133 of the initial plume mass (98.67% reduction), which is only slightly higher than the other designs that are considered in this section.

There are a few reasons that may explain why decreasing the depth of the hydraulic barrier designs considered in this section did not have a larger affect on the plume mass. Heterogeneity, to the degree considered in this heterogeneous conceptual model, results in preferential flow paths that are typically not continuous throughout the model. Having materials with different hydraulic conductivity values creates an effect where flow does not always go in the direction of the steepest gradient, so the travel path of a particle can be extended allowing for more time for decay to occur. Similarly, when impediments to flow are encountered, natural attenuation has more time to degrade the plume mass. Another reason is the anisotropy in the model. Each material is assigned an anisotropy of 3, but there is also an effective anisotropy effect in the model from the difference in hydraulic conductivity of a factor of 100 between the two materials. The low conductivity material acts as a semiconfining unit and significantly reduces transport in the vertical direction caused by a partially penetrating passive hydraulic barrier.
This scenario examines the effect of thickness on a gravel trench. Increasing thickness corresponds to an increase in transmissivity, and an increase in the capture zone of the passive hydraulic barrier. Thus, the anticipated affect of an increase in trench thickness is improved hydraulic isolation of the contaminant source. Scenarios where the trench thickness is 0.61 m (2 ft), 1.22 m (4 ft), and 2.44 m (8 ft) are examined and compared with the case where no trench has been installed. The hydraulic conductivity of the gravel trench is kept constant at 853 m/d. Each design scenario is for a fully penetrating trench with a depth of 15.2 m.

The default gravel trench design (Figure 4.16b and 4.17b), which has a thickness of 0.61 m, reduces plume mass significantly in comparison to the base case scenario (Figure 4.16a and 4.17a). Doubling the thickness of the gravel trench from 0.61 m to 1.22 m results in decreases in the areal and vertical extent of the plume (Figure 4.16c and Figure 4.17c). Figures 4.16d and 4.17d, show the results for a gravel trench with a thickness of 2.44 m, where the plume has been nearly eliminated.
Figure 4.16: Tracer concentrations for different values of trench thickness after 30 years of simulation. The results are for the k=12 plan view. The unit for concentration is mg/L.
Figure 4.17: Tracer concentrations for different values of trench thickness after 30 years of simulation. The cross sections are for i=50. The unit for concentration is mg/L. (Z magnification =3)
The plume mass calculations for each design are summarized in Figure 4.18. A gravel trench with a thickness of 0.61 m experienced a reduction of 99.2% to 0.008 of the initial plume mass. Increasing the thickness of the gravel trench by a factor of two, from 0.61 m to 1.22 m reduced the plume mass by 99.9% to 0.001 of the initial plume mass. Another gravel trench design, where the trench has a thickness of 2.44 m, resulted in a reduction in plume mass of 99.97% to 0.0003 of the initial plume mass. Indeed, Figure 4.18 demonstrates that increasing the thickness of a gravel trench, while maintaining the same hydraulic conductivity (853 m/d) and depth (15.2 m), produces a more effective hydraulic isolation design.

Figure 4.18: Normalized plume mass as a function of time for a heterogeneous conceptual model for different hydraulic barrier thickness values.
Chapter 5

Barra Mansa Site

Hydraulic source zone isolation is being considered at a contaminated site located in Barra Mansa, Brazil. Barra Mansa is located in the Rio de Janeiro province of Brazil and within the Paraiba Valley. It is an important economic region in Brazil due to the level of industrial activity performed there. Industrial activity at this particular site began in 1949 with the production of explosives. By 1978 the capacity of the site had increased and other industrial products were being produced. Some examples of the products include herbicides, pesticides, insecticides, and fungicides, among other products. Some of the chemicals used in the production process include chloroform, carbon tetrachloride, freon, and hydrofluoric acid. The manufacturing of explosives ended in 1996. In 2000, the production of most of the aforementioned chemicals was ended and the primary product of the site was refrigerant gases. In the portion of the site where source zone isolation is being considered, carbon tetrachloride and chloroform leaks were identified in the 1970s. Currently, a series of injection wells are being used to inject amendments into the subsurface and stimulate biodegradation of contaminant plumes at the site. If hydraulic source zone isolation can be successfully implemented, then the injection wells that are in use would no longer be
needed, reducing the cost of remediating the site and providing a more sustainable remediation alternative.

5.1 Location

The location of the industrial site in Barra Mansa that will be the focus of the remaining chapters in this work is shown in Figure 5.1. The focus area for this study is boxed in red in Figure 5.1. This is the area where the injection wells are injecting biostimulants and where carbon tetrachloride and chloroform source zones are suspected of being present. The passive hydraulic barrier is also likely to be installed within the area boxed in red as well. The Paraiba river is labeled and the Goiabal stream outlined in blue. Mountains surround the site and areas that are densely populated with trees in Figure 5.1 represent large, steep hills. The area boxed in red is relatively flat.

Figure 5.1: Map of the Barra Mansa site with the area of focus boxed in red, the Goiabal stream in blue, and the Paraiba river labeled.
5.2 Boundary Conditions

The major river, the rio Paraiba do Sul, and local stream, the Goiabal, are utilized to act as natural boundaries for the model of the site. Due to the lack of another natural boundary, a head contour from a hydraulic head map of the site is used as a general head boundary. The head contour used as the model boundary is highlighted in blue in Figure 5.2. The water level map was made in 2004, which is before injection began at the site. Locations of wells and a carbon tetrachloride plume are also included in Figure 5.2.

Figure 5.2: Barra Mansa site with local stream and contours of hydraulic head from 2004. The 378 m head contour is outlined in bold blue. (DuPont, 2016)
The general head boundary and the Goiabal stream are essentially treated as constant head boundaries due to the high conductance (500 m²/d). The heads assigned at nodes along the Goiabal stream used in this study were obtained from water level measurements taken at the site. The rio Paraiba do Sul is treated as a constant head boundary and assigned a constant head of 375.15 meters. Multiple points along the rio Paraiba do Sul have measured water levels, however, there was great variation in the heads and since the river was large it is treated as having a constant head value along the arc. Since a hydraulic barrier is a permanent structure that will be installed to provide a long term remediation solution, the focus of the groundwater model is on the performance of a hydraulic barrier under average conditions. Consequently, a steady state groundwater flow field is used to assess the hydraulic barriers impact on the flow field over many years. The model boundary conditions are depicted in Figure 5.3.
Figure 5.3: The location and geometry of the model boundary conditions: the general head boundary (red), the river boundary (blue), and the constant head boundary (orange). The length along the stream is 320 m for reference.
5.3 Hydrogeology

The aquifer system consists primarily of undifferentiated fractured crystalline rock. The crystalline rock is Precambrian in age and has a variety of constituents in the region as can be seen in Figure 5.4. Borehole logs show that the bedrock at the focus area of this study is gneiss. Overlying the crystalline rock is unconsolidated regolith. Saprolite makes up part of the regolith from the weathering of gneiss. Above the saprolite are recent alluvial deposits. The alluvial deposits are Quaternary sediments from seasonal flooding of the river and the stream nearby. The alluvium consists of materials such as clay, sand, gravel, and conglomerate. The regolith also include soil which can be above and below the alluvium. The yield of the aquifer varies considerably and is highly dependent on the integrity of the bedrock. Regionally the well yield is poor, however it varies locally depending on the size and the number of fractures a well intersects. The depth to the fractured bedrock is estimated to be about 8 meters below ground level at the site. However not much information is available on the fractured bedrock. Two pumping tests were performed on the aquifer at the site of interest: one in the regolith and another in the fractured bedrock. The estimated hydraulic conductivities of the sedimentary aquifer and fractured bedrock aquifer are 0.08 and 0.80 m/d respectively.
5.4 Recharge

Average yearly rainfall for the past five years in Barra Mansa has been about 1700 mm/yr. The groundwater recharge at the site is unknown. Groundwater recharge estimates are obtained from model calibration. The site consists of both paved surfaces and grassy surfaces. The different surfaces are delineated and used
to create polygons for calibration. To maintain a degree of consistency, the same recharge values are used in all of the aquifer scenarios considered. Recharge values between 0.635 cm/yr (0.25 in/yr) and 10.16 cm/yr (4 in/yr) are assigned to different polygons in the model. Figure 5.5 shows the distribution of the calibrated recharge polygons and the associated values. Higher recharge values were calibrated to grassy surfaces and lower values, such as 0.635 cm/yr, to paved surfaces.
Figure 5.5: Calibrated recharge polygons and associated values. The length along the stream is 320 m for reference.

5.5 Calibration Targets
Thirty-five observation wells are used to calibrate the groundwater model. Calibration targets (Figure 5.6) are calculated by multiplying the z-score for the desired confidence interval by the standard deviation of measured head values to get a value for an interval. The calculated interval is then added and subtracted to the mean observed head to get a calibration target interval. The mean of the heads at each well is assumed to equal the average of the measured heads. Only wells that had a minimum of three measurements within the course of a year were considered. Some of the wells had incorrect elevations at the site and since the water level measurements were given in depth to water values this posed a problem. However, in areas believed to be flat, the elevations were modified and assigned values that compared more favorable with that of nearby wells whose elevations’ are believed to be accurately measured. Injection at the site occurs in intervals and began in January 2011. Since the water levels are significantly altered during injection, only water level measurements taken before 2011 are used to calibrate the groundwater models used in this analysis.
5.6 Discretization

A numerical grid was created that runs about 340 meters along the stream and 532 meters along the river. The vertical thickness of the aquifer system being modeled is 26 meters. The model is divided into 13 layers that are 2 meters thick each. Seven grid refinement points have been included around the source zone to allow for more accurate modeling of contaminant transport near the source. Four grid refinement points, one in each corner of the rectangular shaped hydraulic barrier, are used to allow for the inclusion of a narrow trench in the model. The base size of each of the refining points is 0.2 meters with a bias of 1.1, and a maximum dimension of 4 meters. The result is a model with variable cell dimensions that are as small as 0.1 m x 0.1 m x 2 m and as large as 4 m x 4 m x 2 m. A total of 653,599 active cells are used in the model. The grid generated from the refinement points is shown in Figure 5.7 and it is clear where the focus area for the modeling is. Figure 5.8 shows a zoomed in view of the grid refinement points around the source zone and the grid refinement points used for the corners of the permeable hydraulic barrier. The model extends along the river away from the refined cells so that the convergence of the head contour and the constant head boundary is sufficiently far enough away from the model so that it does not affect the simulation results.
Figure 5.7: An areal view of the entire grid after refinement. The length of the model along the stream is 320 m for reference.
5.7 Transport Conceptual Model

Source zone and plume delineation efforts are ongoing at the site, however to demonstrate the feasibility and performance of a hydraulic barrier an idealized source zone is created in order to proceed with the modeling effort. The source will be treated as a conservative tracer with a constant source zone concentration of 10,000
ppb. The areal extent of the source zone used in the model is based on a 2004 contour map of a contaminant at the site (Figure 5.9). The source zone has an area of 709 m².

Figure 5.9: Concentration contour map from 2004 with legend for chloroform at the site. The highest concentration contour on the map is used to create the areal extent of the idealized source zone used in this work. (DuPont, 2016)

The effects of decay due to natural attenuation and adsorption onto the aquifer matrix are not considered for two reasons. One reason is that emphasis is placed on the impact that source zone isolation on the hydraulics of the site since that will govern how well it performs. The other reason is that more information is needed on the extent of the contamination, particularly recent estimates of depth and areal extent. With its true depth unknown, the tracer source is assigned to all model layers above the bedrock or fractured bedrock. The calibrated groundwater flow fields with a source present are used to generate a plume using MT3DMS. The base case transport simulations are run for 30 years because the leaks were identified at
the site were identified around that time.

Figure 5.10: The source zone is depicted and indicated by the arrow as the purple region. The source zone is assigned a constant concentration of 10,000 ppb.

5.8 Hydraulic Isolation Designs

The hydraulic barrier is included by creating a rectangular region of uniform depth and thickness around the contaminant source zone. The thickness of the hydraulic barrier considered in these simulations is 0.80 m. The dimensions of the hydraulic barrier is 53 m x 38 m in all of the cases considered. The hydraulic barrier fully penetrates the aquifer, but is not present in layers that represent bedrock or
fractured bedrock. The designs considered are a gravel trench, a gravel trench with pipe, and a gravel trench with pipe and a source zone extraction well. The effective hydraulic conductivity of the gravel trench is 853 m/d (2,800 ft/d). The design with pipes and a gravel trench is such that it can be represented by an effective hydraulic conductivity of 8530 m/d (28,000 ft/d). The location of the passive hydraulic barrier at the site is shown in Figure 5.11. A sketch of the hydraulic barrier inclusion and construction to bedrock is shown in Figure 5.12.
Finally, a design that includes the gravel trench and pipe and also incorporates low flow source zone pumping is considered for certain scenarios as well. The extraction well no longer makes the hydraulic isolation system passive, due to the continuous pumping and treatment of the contaminated water, but is a design that should be investigated as well. With the inclusion of a hydraulic barrier to neutralize the head gradient across a source zone, low flow rates can be used to prevent the vertical advective transport effect in the source zone for a partially penetrating or equivalent design scenario. The source zone pumping design considered includes a well that is screened at the bottom layer of the aquifer and pumps at a rate of 0.278 L/min (0.073 gpm). A low flow rate was selected due to the difficulty in getting a solution to converge for higher flow rates. This design was selected over the design that included wells in the gravel trench because source zone pumping can create an inward and upward gradient that counteracts the downward flow component that may occur in certain hydrogeologic scenarios such as a contaminated aquifer under-
lain by fractured bedrock. If pumping and injection wells are used in the trench, the effect in Figure 3.23 could hamper the hydraulic isolation effort. Figure 5.13 shows the location of the pumping well relative to the source zone (pink) and the hydraulic barrier (red). The simulations that include hydraulic barrier designs are run for 60 years in order to allow the contaminant transport model to reach steady state or near steady state conditions.

Figure 5.13: The hydraulic barrier and a plan view of the location of the source zone pumping well (white circle).
Chapter 6

Intact Bedrock Scenario

The remaining chapters in this thesis will explore the performance of hydraulic source zone isolation designs under different hydrogeologic conditions. The first scenario that will be investigated is one where the bedrock is treated as having a low effective hydraulic conductivity and is considered as being mostly intact. The calibrated hydraulic conductivity zones used for the groundwater model are shown in Figure 6.1 and 6.2.

The range of hydraulic conductivity values used fall within the range of estimates of hydraulic conductivity obtained from slug tests. Most of the aquifer is close to the hydraulic conductivity of 0.08 m/d (green), which was obtained from a pumping test performed at the site. Higher hydraulic conductivity values were calibrated closer to the river to get the calculated heads to match better with the observed heads. A fairly low hydraulic conductivity of $8 \times 10^{-4}$ m/d was assigned to the bedrock.
Figure 6.1: Calibrated hydraulic conductivity polygons for the intact bedrock case. The hydraulic conductivities shown here correspond to aquifer layers 1-7 of the model. The anisotropy of the aquifer is 10. The units for the legend are m/d. The length along the stream is 320 m for scale.
Figure 6.2: The hydraulic conductivity of the bedrock is $8 \times 10^{-4}$ m/d. The bedrock is treated as being isotropic and the units for the legend are m/d. The length along the stream is 320 m for scale.
6.1 Groundwater Flow Calibration

The targeted calibration intervals were mostly met at locations where observed head measurements were available. The mean residual for the heads is 0.08 m and the mean absolute residual 0.24 m. A root mean squared residual error of 0.36 m is calculated for this scenario. The normalized root mean squared error, which is obtained by dividing the root mean squared residual by the range of observed heads, is 0.104. In Figure 6.3, the largest error was over 200% and had a red error bar. The location of that measurement is not located in the area where the focus hydraulic isolation analysis will take place so it is likely to not have a significant affect on the simulation results. Figure 6.4 provides a closer look at the location where the hydraulic barrier will be included and confirms that in the area of greatest importance in the model, the groundwater calibration is acceptable and the calibration target intervals are generally met. Figure 6.5 shows that there is an acceptable amount of agreement between the computed heads and the observed heads for this scenario.
Figure 6.3: The groundwater head contours and model calibration results for the intact bedrock scenario. The calibration targets are generally met throughout the model. The unit for the legend is meters. The length along the stream is 320 m for scale.
Figure 6.4: An enlarged view of the error bars in the focus area for this analysis with the intact bedrock scenario results. All but two of the twenty-nine calibration target intervals in this area are met. The unit for the legend is meters.
6.2 Contaminant Transport

The contaminant transport simulation shows the plume flowing towards the river which is consistent with site observations. There is also a downward component of flow that is transporting the plume into layers where the tracer was originally not present. In Figures 6.6a and 6.6b, the plume is shown in layer 6 and layer 7 of the contaminant transport simulation after 30 years. The time frame of 30 years is selected because some of the contaminants present at the site were identified around that time. The effects of the hydraulic conductivity polygons on the transport of the plume is evident in Figures 6.6a and 6.6b, where the plume has two fronts that have formed as a result of preferential flow through polygons with higher hydraulic conductivity. The vertical transport into layer 7 is shown in the cross sectional slice shown in Figure 6.6c. Only small concentrations of the tracer are present in the bedrock due to its low hydraulic conductivity.
After 30 years of simulation, the contaminant transport simulation has yet to reach a steady state. Instead, the plume mass continues to increase. This behavior is captured in Figures 6.6a and 6.6b, where the plume has not made contact with the river boundary or the stream. The plume mass plot shown in Figure 6.7 depicts this effect more clearly. The plume mass is still rising and the slope has not decreased to indicate interaction with the model boundaries. The plume concentration at 30 years will be used as the starting concentration for the transport simulations that account for the inclusion of a hydraulic isolation design.
Figure 6.6: Base case plume concentrations after 30 years of simulation for the intact bedrock scenario. The unit for concentration is parts per billion (ppb). The length along the stream is 320 m for scale.
6.3 Gravel Trench Results

The gravel trench design considered for this scenario reduced plume concentrations noticeably as a result of the reduction in head gradient across the source zone. Figure 6.8 shows the effect of the gravel trench design on the existing plume and the source zone after 60 years of simulation. Layer 6 in Figure 6.8a and layer 7 in Figure 6.8b both show relatively high concentrations of tracer breaching the gravel trench. Figure 6.8c confirms that a fairly high tracer concentration is leaving the source zone and being transported vertically, towards bedrock. Despite these effects, the addition of a gravel trench has reduced the long-term plume mass from an initial value of 11.1 kg to 2.0 kg, a reduction of 82%.

Figure 6.7: Plume mass as a function of time for the base case condition of the intact bedrock scenario.
Figure 6.8: Contaminant concentrations after 60 years of simulation with the inclusion of a gravel trench design for the intact bedrock scenario. The units for concentration are ppb. The length along the stream is 320 m for scale.
Although, this result is a bit misleading. The plot of plume mass in Figure 6.9 has two curves. One curve represents the case where the source is exposed to the natural groundwater gradient of the site (blue curve), and another that represents plume mass after installation of a gravel trench (orange curve). Under natural conditions for this scenario, the plume does not reach the model boundary until 45 years, after which the rate that plume mass increases begins to decrease and the system appears to be nearing steady state at 90 years with a plume mass of 25 kg. Since the plume has yet to interact with the model boundaries when the gravel trench simulation begins, there is initially a slight increase in plume mass after the inclusion of the gravel trench at 30 years. At about 32 years the plume mass levels off, indicating interaction with the river. After about 40 years, plume mass begins to decrease at an increasing rate until 60 years, at which point the system begins to approach steady state which is reached near 80 years.

Figure 6.9: Plume mass as a function of time comparing the plume mass without a hydraulic barrier (blue) to the plume mass when a gravel trench is included after 30 years (orange).
6.4 Gravel Trench with Pipes Results

The inclusion of a gravel trench with pipe (K=8,530 m/d) improves the hydraulic source zone isolation efforts. The gravel trench with pipe design, like the gravel trench design, has not been able to completely detach the plume from the source zone. Despite this lack of total hydraulic isolation, the plume mass is reduced from its starting mass of 11.1 kg to 1.74 kg, a reduction in plume mass of 85%. The transport simulation results show an improvement when compared to the results shown in Figure 6.8. Figure 6.10 shows the contaminant transport results after 60 years of simulation for a design that includes pipe in it and has an effective hydraulic conductivity of 8,530 m/d.
Figure 6.10: Contaminant concentrations after 60 years of simulation with the inclusion of a gravel trench with pipe design for the intact bedrock scenario. The units for concentration is ppb.
The response of the aquifer to perturbation when a gravel trench with pipe design is included is nearly the same as the aquifer response to a gravel trench. Plume mass increases briefly before leveling off and decreasing at a slow rate until 40 years. The increase in plume mass takes place for nearly 2 years, at which point the plume early interaction with the model boundaries. The response time of the aquifer to the perturbation it undergoes after installation of the gravel trench with pipe design is 50 years since it reaches a steady state at 80 years.

![Figure 6.11: Plume mass as a function of time comparing the plume mass without a hydraulic barrier (blue) to the plume mass when a gravel trench with pipe is included after 30 years (orange).](image)

### 6.5 Comparison

The plume mass reductions from the starting concentrations of both of the designs (the gravel trench and gravel trench with pipe) were considered in this chapter and are compared in Figure 6.12. It is clear that a greater effective hydraulic conductivity for the hydraulic barrier will result in a larger reduction in plume mass for this scenario. By reexamining Equation (2.8), we see that this can be achieved by increasing the number of pipe or increasing the diameter of the pipe in the gravel
trench. The additional 3% reduction in plume mass when scaled by the initial concentration may or may not be significant, depending on the targeted MCL, however, it can be improved by increasing the effective hydraulic conductivity of the trench.

Figure 6.12: Comparison of plume mass over time for the gravel trench (green) and gravel trench with pipe (blue) designs for the intact bedrock scenario.
A scenario where a highly conductive layer intersects the center of the aquifer is considered. The aquifer is present in the first 6 layers of the model. The model has a top elevation of 386 m and the aquifer reaches bedrock at 372 m. Bedrock is represent by layers 7 through 13 and has a bottom elevation of 360 m. Layer 4, which is present in the interval 380 m to 378 m of the aquifer is treated as the highly conductive bed and has a hydraulic conductivity of 3 m/d. The distribution of the calibrated hydraulic conductivity values throughout the aquifer is shown in Figures 7.1, 7.2, and 7.3.
Figure 7.1: Hydraulic conductivity calibration polygons used for the aquifer layers 1-3 and 5-6 for the highly conductive layer scenario. The anisotropy of each of the aquifer layers is 10. The units for the scale are m/d. The length along the stream is 320 m for scale.
Figure 7.2: The hydraulic conductivity of the highly conductive bed at 3 m/d. The layer has an anisotropy of 10 and the units for the scale are m/d. The length along the stream is 320 m for scale.
7.1 Groundwater Flow Calibration

The mean residual for the calibrated heads is -0.05 m and the mean absolute residual the heads is equal to 0.25 m. The root mean squared residual is 0.31 m and a normalized root mean squared residual of 0.09 is calculated for this calibration which is acceptable for the purposes of this analysis. Figure 7.4 shows that only five out of

Figure 7.3: The hydraulic conductivity of the bedrock as $8 \times 10^{-4}$ m/d. The bedrock is treated as being isotropic and the units for the scale are m/d. The length along the stream is 320 m for scale.
the thirty-five wells considered are not green and do not fall within the target interval. These wells tend to be located near the boundaries of the model and are generally away from the area of focus of the simulations. An enlarged view of the focus area in Figure 7.5 shows one well where the calculated head does not fall within the target interval and that well is located near the stream boundary (Figure 7.5). The plot of the computed heads and observed heads in Figure 7.6 also shows an acceptable degree of agreement between the two values.

Figure 7.4: The groundwater head contours and model calibration results for the highly conductive layer scenario. Thirty out of the thirty five wells used for model calibration meet the calibration target interval. The scale is in meters. The length along the stream is 320 m for scale.
Figure 7.5: The groundwater head contours and error bars where the source zone and hydraulic barrier will be included for the highly conductive layer scenario. The scale is in meters.
7.2 Contaminant Transport

The contaminant transport simulation results after 30 years of simulation show that plume trajectory is in the direction of the river. This result is shown in Figure 7.7. Unlike the intact bedrock scenario, the plume seems to have one leading front. Most of the transport through the aquifer occurs through the highly conductive layer but because the plume has a downward flow component, the aquifer layers beneath the highly conductive layer 4 also transport a significant amount of the tracer. There is some transport occurring within the upper layer of the bedrock, however, it is not a significant amount since the bedrock is assigned a low hydraulic conductivity. Although the plume has reached the model boundary after 30 years, as shown in Figures 7.7a, 7.7b, and 7.7c, steady state for the transport simulation has not yet been reached. Figure 7.8 confirms that steady state has not yet been reached because the plume mass is still increasing. After about 4 years plume begins to interact with the model boundaries. The plume mass after 30 years of simulation for the base case (no hydraulic barrier) is 31.5 kg. This is a much higher plume mass in comparison to
the intact bedrock scenario due to the transport taking place in the highly conductive layer 4.

Figure 7.7: Base case contaminant concentrations results after 30 years of simulation for the highly conductive layer scenario. The scale has units of ppb. The length along the stream is 320 m for scale.
7.3 Gravel Trench Results

Upon inclusion of the gravel trench with a hydraulic conductivity of 853 m/d, the plume mass decreases significantly due to the flushing out of the existing plume with uncontaminated, upstream groundwater and a significant reduction in mass discharge out of the source zone. Despite the reduction of plume mass shown in comparison to the base case, there is still room for improvement. Figure 7.9a and 7.9c in particular show that there remain concentrations that are quite large in the highly conductive layer in the aquifer after 60 years of simulation. The contrast in hydraulic conductivity between the highly conductive layer and the gravel trench should be higher in order to reduce the plume mass through that layer by an order of magnitude.
Figure 7.9: Concentration distribution after 60 years upon inclusion of the gravel trench for the highly conductive layer scenario. The concentrations are in units of ppb. The length along the stream is 320 m for scale.
The reduction in plume mass due to the inclusion of the gravel trench is initially very rapid from 30 to 32 years (Figure 7.10), which is due to the increase in the rate of plume mass exiting the model. After 32 years, the rate at which plume mass decreases begins to gradually decline due to the reduction in the mass load in the plume, which results in less tracer mass exiting the model due to source zone isolation. A steady state is reached at about 85 years. Figure 7.10 shows that after 5 years the plume mass is nearly halved from 31.5 kg to 16.7 kg. After simulating the effect of the inclusion of the gravel trench for 60 years, the plume mass is reduced to a concentration of 4.2 kg, a reduction of 87%.

![Figure 7.10: Plume mass as a function of time comparing the plume mass without a hydraulic barrier (blue) with the plume mass if a gravel trench (orange) is included after 30 years.](image)

**7.4 Gravel Trench with Pipes Results**

For a design that includes pipes in addition to a gravel trench (hydraulic conductivity of 8530 m/d), the 60 year hydraulic source zone isolation simulation results
are more promising. Figure 7.11 shows that the contrast in hydraulic conductivity between the highly conductive aquifer layer and the hydraulic barrier is sufficient to reduce the tracer concentrations transported by an order of magnitude in comparison to the gravel trench design. Source zone isolation has been improved due to the increase in effective hydraulic conductivity. The plume has been detached from the source in some areas in layer 6 of the model as shown in Figure 7.11b.

The initially rapid reduction in plume mass at early times for the gravel trench with pipe curve is steeper than the results in Figure 7.10 for a gravel trench which indicates that mass is exiting the model at a faster rate. After 10 years, the plume mass is reduced to 11.9 kg from 31.5 kg and after 60 years the plume mass is reduced to 1.78 kg, a reduction of 94%. Steady state appears to be reached at 85 years
Figure 7.11: Concentration distribution after 60 years of simulation after inclusion of the gravel trench with pipe design for the highly conductive layer scenario. The units for concentration are in ppb. The length along the stream is 320 m for scale.
7.5 Gravel Trench with Pipes and Source Zone Pumping Results

A low flow source zone pumping well is included in addition to the gravel trench with pipes. The flow rate of the extraction well is 0.4 m$^3$/d or 0.073 gpm. The distribution of the tracer has been noticeably altered and the source is nearly completely isolated as result of the inward gradient caused by source zone pumping (Figure 7.13). Figure 7.14 shows that the plume mass was reduced from 31.5 kg to 1.41 kg. In comparison to the gravel trench with pipe design, the plume mass is reduced by 0.37 kg. This simulation reaches steady state at about 81 years.
Figure 7.13: Concentration distribution after 60 years upon inclusion of the gravel trench with pipe and source zone pumping design for the highly conductive layer scenario. The units for concentration are ppb. The length along the stream is 320 m for scale.
Figure 7.14: Plume mass as a function of time comparing the plume mass without a hydraulic barrier (blue) with the plume mass if a gravel trench with pipe and source zone pumping (orange) is included after 30 years.
7.6 Comparison

Plume mass was noticeably reduced for the gravel trench design, but there was still a noteworthy amount of tracer being transported through the highly conductive bed. If pipe is included in the gravel trench and accounted for by increasing the effective hydraulic conductivity of the barrier from 853 m/d to 8530 m/d, then there is a reduction in plume mass from the 4.1 kg achieved using a gravel trench to 1.78 kg when a design that includes pipe in the gravel trench is used. The transport of the tracer through the highly conductive layer is reduced by an order of magnitude as well. If low flow source zone pumping is performed, then the plume mass is reduced even further to 1.41 kg. This is may not seem like a significant difference in comparison with the gravel trench with pipe design simulations, but as shown in Figure 7.13, the plume is nearly completely detached from the source. There is also a slight reduction in tracer mass in the bedrock material. The mass in the bedrock can be further reduced by increasing the source zone pumping rate. Simulations that used source zone pumping wells with higher extraction rates were attempted, however as the flow rate increased, the solutions proved difficult to converge and showed signs of numerical instability.
Figure 7.15: Plume mass as a function of time for the inclusion of a gravel trench (green), gravel trench with pipe (blue), and source zone pumping (yellow).
Chapter 8

Fractured Bedrock Scenario

A case where the bedrock is highly fractured is now considered. The calibrated hydraulic conductivity distributions are shown in Figures 8.1, 8.2, and 8.3. Layers 1 through 6, which span 385 m to 374 m of the model, represent the aquifer and the calibrated hydraulic conductivity values for those polygons are shown in Figure 8.1. Layer 7, which spans 374 m to 372 m of the aquifer model, is used to represent a fractured bedrock layer and the calibrated hydraulic conductivity values for those polygons are shown in Figure 8.2. The bedrock, is represented in layers 8 through 13 of the model and corresponds with depths of 372 m to 360 m. The hydraulic conductivity of the bedrock material is $8 \times 10^{-4}$ m/d. The aquifer material is assigned an anisotropy of 10 and the fractured bedrock and bedrock layers are treated as being isotropic. The hydraulic conductivity throughout most of the fractured bedrock in Figure 8.2 is around 0.8 m/d, which is similar to the pumping test derived hydraulic conductivity estimate for the fractured bedrock at the site.
Figure 8.1: Calibrated hydraulic conductivity polygons for aquifer layers 1-6 of the fractured bedrock case. The anisotropy of each of the aquifer is 10. The units for the scale are m/d. The length along the stream is 320 m for reference.
Figure 8.2: The calibrated hydraulic conductivity polygons for the fractured bedrock aquifer layer 7. The fractured bedrock is assumed to be isotropic. The units for the scale are m/d. The length along the stream is 320 m for reference.
Figure 8.3: The hydraulic conductivity of the bedrock as $8 \times 10^{-4}$ m/d. The bedrock is treated as being isotropic and the units for the scale are m/d. The length along the stream is 320 m for reference.
8.1 Groundwater Flow Calibration

Error bars for the computed heads nearly all meet the calibration target interval and indicate good agreement with the observed heads. The local heads in the region where the tracer source zone and hydraulic barrier are included in the model all fall within the targeted calibration interval with the exception of one well which is close to the stream boundary as shown in Figure 8.5. A small mound can be seen in Figures 8.4 and 8.5 which is a result of the high recharge in that zone (see Figure 5.5). The mean residual for the heads is 0 m and the absolute residual the heads is equal to 0.21 m, which are less than the values calculated for the previous models considered up until this point. The root mean squared residual error of 0.30 m is also the least from the cases considered thus far. A normalized root mean squared of 0.086 is calculated for this scenario which is acceptable. The plot of computed heads and observed heads in Figure 8.6 also reaffirms that the model has been sufficiently calibrated for the goals of this project.
Figure 8.4: The groundwater head contours and model calibration results for the fractured bedrock scenario. The unit for the legend is meters. The length along the stream is 320 m for scale.
Figure 8.5: The groundwater head contours and error bars where the source zone and hydraulic barrier will be included for the fractured bedrock scenario. The unit for the legend is meters.
Figure 8.6: Plot of the computed heads vs the observed heads for the fractured bedrock scenario. The units for head along both axes is meters.
8.2 Contaminant Transport

The plume generated from the groundwater flow field is traveling towards the river, which is consistent with what has been observed at the site. The planar view contour maps in Figure 8.7a and 8.7b show high concentrations being transported through those layers. The vertical cross section in Figure 8.7c shows that a significant fraction of the mass of the plume is traveling through the fractured layer as opposed to the aquifer. The preferential flow towards and through the fractured layer is a result of the hydraulic conductivity contrast between the aquifer and the fractured bedrock.

Fractured bedrock complicates flow fields and in this case, where it is represented with an equivalent porous medium, its effect is simplified to a certain degree. Nevertheless, the general effect at a similar site will be similar, a downward component of groundwater flow and contaminant transport due to the fractured aquifer having a higher hydraulic conductivity than the overlying aquifer. In Figure 8.8, plume mass is calculated and the contaminant transport simulation for this scenario appears to be nearing steady state at 30 years, which indicates that plume mass is exiting the model through the model boundaries. At about 15 years, the plume begins to interact with the model boundary and reaches steady state after 33 years (Figure 8.10).
Figure 8.7: Base case tracer concentrations after 30 years for the fractured bedrock scenario. The units for the legend are in ppb. The length along the stream is 320 m for scale.
Figure 8.8: Plume mass as a function of time for the fractured bedrock base case.
8.3 Gravel Trench Results

The contaminant transport results after 60 years of simulation are shown in Figure 8.9. The mass exiting the source zone has been reduced. The presence of the gravel trench hydraulic barrier (K=853 m/d) is felt in the aquifer and the source is somewhat isolated. Additionally, in the fractured bedrock layer, the presence of the hydraulic barrier is felt despite the hydraulic barrier not being installed through any bedrock. Despite the reduction in concentration, there is still a significant amount of transport taking place through the fractured layer.

Due to the fractured bedrock layer, this system reaches steady state at 53 years (Figure 8.10) which as faster than the other scenarios considered which reached steady state at around 80 (highly conductive layer case) or 90 years (intact bedrock case). The inclusion of a gravel trench results in a decrease in plume mass from 9.5 kg to 2.0 kg, a reduction of 79%. The 79% reduction in plume mass is significant, but it is evident that the source zone is not as effectively isolated as the previously tested scenarios due to the limited influence the gravel trench has on tracer mass that is in the fractured layer and the vertical flow that is induced due to the presence of fractured bedrock.
Figure 8.9: Tracer concentrations after 60 years of simulation with the inclusion of a gravel trench for the fractured bedrock scenario. The units for the legend are in ppb. The length along the stream is 320 m for scale.
Figure 8.10: Plume mass as a function of time comparing the plume mass without a hydraulic barrier (blue) with the plume mass if a gravel trench is included after 30 years (orange) for the fractured bedrock scenario.
8.4 Gravel Trench with Pipe Results

The inclusion of a gravel trench with pipe (K=8,530 m/d) after 60 years results in a concentration distribution that is similar to the concentration distribution of the results from the gravel trench simulations. The plume concentration distribution shown in Figure 8.11 is nearly identical to the gravel trench design results in Figure 8.9. The fractured layer is still transporting the majority of the tracer, hampering the affect of this hydraulic barrier design. The plume mass plot in Figure 8.12 shows that plume mass is reduced from 9.5 kg to 1.9 kg, a reduction of 80%. This demonstrates hardly an increase in comparison to the gravel trench design in the previous section. The limitations of increasing the effective hydraulic conductivity alone are demonstrated here. The time to steady state is 23 years (the system reaches steady state at 53 years), which is also the same for the gravel trench design.
Figure 8.11: Tracer concentrations after 60 years upon inclusion of the gravel trench with pipe design for the fractured bedrock scenario. The units for the legend are in ppb. The length along the stream is 320 m for scale.
Figure 8.12: Plume mass as a function of time comparing the plume mass without a hydraulic barrier (blue) with the plume mass if a gravel trench with pipe is included after 30 years (orange) for the fractured bedrock scenario.

8.5 Gravel Trench with Pipe and Source Zone Pumping Results

When a source zone pumping well is added to a gravel trench with pipe design and the extraction well is screened above the fractured layer in layer 6 of the aquifer (376 m to 374 m), an improved degree of source zone isolation is obtained as shown in Figure 8.13. The extraction rate is 400 L/d (0.07 gpm), and creates enough of an inward and upward gradient that a fairly significant amount of tracer is removed from the fractured layer. The well is screened in layer 6 of the model, however presence is felt in the fractured bedrock layer beneath it. A visual comparison of Figure 8.13b and Figures 8.11b and 8.9b shows that the source zone pumping taking place in layer 6 in the model, has an affect on plume mass concentrations by the river.
Figure 8.13: Tracer concentrations after 60 years upon inclusion of the gravel trench with pipe and source zone pumping design for the fractured bedrock scenario. The units for the legend are in ppb. The length along the stream is 320 m for scale.

The response to source zone pumping results in a steeper initial decline in plume mass is observed, although steady state is reached at about 54 years which is similar to the gravel trench with pipe design. Figure 8.14 shows the reduction in plume mass reduced from 9.5 to 1.3 kg, a reduction of 86%. The effect of the inward...
and upward gradient is felt and mass discharge into and through the fractured layer decreased.

Figure 8.14: Plume mass as a function of time comparing the plume mass without a hydraulic barrier (blue) with the plume mass if a gravel trench with pipe and source zone pumping is included after 30 years (orange) for the fractured bedrock scenario.

8.6 Comparison

Upon comparison of the gravel trench and the gravel trench with pipe design, it is evident that increasing the effective hydraulic conductivity of the hydraulic barrier will not make any significant changes to the plume mass for a scenario where the bedrock is fractured. Source zone pumping will likely be needed if the bedrock is fractured since most of the transport of the tracer may be taking place within the fractured layer. Figure 8.15 shows a comparison of the plume mass over time for the different hydraulic barrier designs considered in this chapter. Although the presence of the hydraulic barrier designs considered in this chapter is felt in the fractured layer and there is a plume mass reduction of about 80% for permeable hydraulic barrier designs. Source zone pumping will be needed to reduce the plume mass further and reduce the tracer mass in the fractured bedrock layer. Higher pumping rates were
not included due to the difficulty in getting the numerical solution to converge.

Figure 8.15: Plume mass as a function of time with the inclusion of a gravel trench (blue), gravel trench with pipe (orange), and source zone pumping (gray).
Chapter 9

Heterogeneous Intact Bedrock Scenario

A scenario that incorporates a greater degree of heterogeneity is generated where borehole logs from the Barra Mansa site are used to model the hydrostratigraphy in greater detail. Boreholes from the site are generalized to include five materials. The five materials consist of alluvial deposits such as fine sand, clean sand, and clay and the metamorphic materials such as saprolite, and bedrock. An example of a borehole that samples all of the materials is shown in Figure 9.1. Although there are many boreholes at the site, Figure 9.2 shows that only 12 are located within the prescribed boundaries of the model. Cross sections were created by matching borehole horizons in GMS and the Solids module was used to generate a 3D representation of the aquifer system. The Solids model is used to estimate the location of the alluvial material and saprolite contact as well as the saprolite and bedrock contact.
Figure 9.1: Example of a borehole from the site

Figure 9.2: Spatial locations of borehole logs
To account for the heterogeneity of the alluvial deposits, T-Progs was used to create a realization of the fine sand, clean sand, and clay distributions. The T-Progs generated distribution was then used to overlay the saprolite and bedrock units from the Solids model and the final depiction for this case is shown in Figure 9.3. This was done to preserve the layering of the system, which is a strength of the Solids module, and incorporate heterogeneity in the alluvial material, which is a strength of T-Progs.

Figure 9.3: Heterogeneous intact bedrock scenario spatial distribution of materials
9.1 Groundwater Flow Calibration

The groundwater flow calibration is very important for validating the numerical groundwater model. However, with the information available for the site, emphasis is placed more on the hydraulics of the problem and the performance of the different hydraulic barrier designs. Therefore, only one realization is used to model the heterogeneity in the alluvial deposits at the site. While the residuals between the calculated and measured heads were considered, attention was also paid to the trajectory of the plume. The calibrations with greatest agreement with the calculated and observed heads gave misleading plume trajectories. The final model parameters selected for this scenario for the calibration was a compromise between residual error and the plume trajectory. To do so, the conductance of the stream was decreased in order to direct the plume towards the river and the hydraulic conductivity of the materials adjusted to get an acceptable level of agreement between the calculated heads and the observed heads. In order to get more flow towards the river, the horizontal anisotropy (K_y/K_x) was decreased to 0.3 so that the plume would preferentially flow in the x direction in the model which is perpendicular to the river. The conductance of the stream was also significantly reduced from 500 m² d⁻¹ to 0.1 m² d⁻¹ in order to have a plume trajectory that reflects what is anticipated at the site more accurately. The calibrated properties are shown in Figure 9.4 below.

![Figure 9.4: Calibrated material properties for heterogeneous intact bedrock scenario.](image-url)
The mean residual was -0.37 m and the mean absolute residual 0.46 m. The root mean squared residual was calculated to be 0.50 m. The normalized root mean square error is 0.144, which is greater than the desirable result of less than 0.10, is acceptable for the purposes of this analysis. The calibration summary statistics that have been computed are not indicative of a good calibration for this scenario, but the point of testing this case is to see how a hydraulic barrier performs in the presence of a higher degree of heterogeneity and with an intact bedrock. Despite the modest calibration summary statistics, Figures 9.5 and 9.6 shows that most of the target calibration intervals are met and that most of the errors are located near the model boundaries with the exception of one well (Figure 9.6). Figure 9.7 shows the agreement between the calculated and observed heads and it is clear that the the level of agreement in this scenario is not as high as in the scenarios examined in previous chapters, but is sufficient for the purposes of this demonstration.
Figure 9.5: The groundwater head contours and model calibration results for the heterogeneous intact bedrock scenario. The heads are measured in meters and the distance along the stream is 320 m for scale.
Figure 9.6: The groundwater head contours and calibration error bars zoomed into the area of greatest importance in the model for the heterogeneous intact bedrock scenario. The heads are in meters.
Figure 9.7: Plot of the computed heads vs observed heads for the heterogeneous intact bedrock scenario. The units along both of the axes is meters.

The plume trajectory in Figure 9.8 shows the plume going towards the mouth of the stream. This is not ideal, but for this scenario, a compromise needed to be met between the plume trajectory and the model calibration. There is a significant amount of transport through the saprolite layer, above the bedrock. This is because at 0.5 m/d, the saprolite is the highest conductivity material with the greatest amount of continuity in this model scenario.
Figure 9.8: Concentration distribution after 30 years of transport for the base case heterogeneous intact bedrock scenario. The concentration units are in ppb and the distance along the stream is 320 m for scale.

9.2 Gravel Trench Results

A visual comparison of Figures 9.8 and 9.9 show the distinction between the plume mass for the base case scenario and a case where a gravel trench (hydraulic conductivity of 853 m/d) is included. In Figure 9.9, there is much less mass discharging through the source zone. Despite the reduction in plume mass, the gravel trench
design’s performance does leave room for improvement. Figure 9.9b shows that there is some mass discharging from the source in the saprolite layer at relatively high concentrations. Upon inclusion of the gravel trench, there also appears to be enhanced vertical transport of the plume beneath targeted isolation area (Figure 9.9c). The hydraulic conductivity used for the portion of bedrock included in the aquifer model does accommodate flow and the tracer is hydraulically transported downward and through the poorly conductive layer. The cause is a downward hydraulic gradient created between the hydraulic barrier and the bedrock material. Since the gravel trench is not installed in the bedrock and the bedrock is not impermeable, on the downgradient dimensions of the hydraulic barrier, there is a hydraulic gradient that generates the downward transport of the tracer component. Figure 9.10 shows that the plume mass gets reduced from 41.7 kg to 3.11 kg after 60 years of simulation, a reduction of 93%. This high reduction rate is encouraging and demonstrates how robust a simple gravel trench design is in the presence of site heterogeneity. After 25 years of simulation, the base case plume begins to interact with the model boundaries and reaches steady state at 70 years. The response time to perturbation of the system from the inclusion of the gravel trench occurs at 50 years.
Figure 9.9: Concentration distribution after 60 years of simulation upon inclusion of the gravel trench (K=853 m/d) for the heterogeneous intact bedrock scenario. The concentration units are in ppb and the stream has a length of 320 m for scale.
Figure 9.10: Plume mass as a function of time for the case where no hydraulic barrier is installed (blue) and a case where inclusion of a gravel trench (K=853 m/d) occurs after 30 years (orange).
9.3 Gravel Trench with Pipe Results

Figure 9.11 shows that there is a significant reduction in plume mass due to the greater degree of source zone isolation achieved. If pipe is included in the gravel trench (K=8530 m/d) and a contaminant transport simulation is run for 60 years, then the all around performance of the hydraulic barrier is improved. Figure 9.12 displays a reduction in plume mass from 41.7 kg to 1.8 kg, a reduction in 96%. However, there is also some vertical transport of the tracer into the bedrock material Figure (9.11c). Although the plume mass has been significantly reduced and the source isolated to a greater degree, it is clear that increasing the effective hydraulic conductivity of the barrier does not address the issue of vertical transport of the tracer. There is still a slight downward hydraulic gradient created along the downstream dimensions of the hydraulic barrier.
Figure 9.11: Concentration distribution after 60 years of simulation upon inclusion of the gravel trench with pipe (K=8530 m/d) for the heterogeneous intact bedrock scenario. The concentration units are in ppb and the stream has a length of 320 m for scale.
Figure 9.12: Plume mass as a function of time for the case where no hydraulic barrier is installed (blue) and a case where inclusion of a gravel trench with pipe occurs after 30 years (orange).
9.4 Gravel Trench with Pipe and Source Zone Pumping Results

A design that now includes source zone pumping at a rate of 400 L/d (0.07 gpm) in addition to a gravel trench with pipe is considered and the transport simulation results shown in Figure 9.13. After simulating the transport of the tracer for 60 years, Figure 9.14 shows that there is a reduction of plume mass from 41.7 kg to 0.64 kg, a reduction of 98.5%. The creation of an inward and upward hydraulic gradient from the pumping well has worked to counteract the downward hydraulic gradient caused by inclusion of the passive hydraulic barrier. Lower concentrations are now present in the bedrock, and the source is nearly completely detached from the plume (Figure 9.13).
Figure 9.13: Concentration distribution after 60 years upon inclusion of the gravel trench with pipe (K=8530 m/d) and a low flow source zone pumping well (Q=0.278 L/min) for the heterogeneous intact bedrock scenario.
Figure 9.14: Plume mass over time with and without the inclusion of a gravel trench with pipe and source zone pumping for the heterogeneous intact bedrock scenario
9.5 Comparison

Each design results in a significant reduction in plume mass. The gravel trench, gravel trench with pipe, and gravel trench with pipe and source zone pumping designs each resulted in a decrease in plume mass from 41.7 kg to 3.1 kg, 1.8 kg, and 0.64 kg or a 93%, 96%, and 98.5% reduction respectively. When comparing the simulation results for this scenario (Figure 9.15), the vertical transport of the tracer is something that cannot be ignored. Although the differences in the plume mass reduction do not seem large, the vertical transport of the tracer into bedrock could make long term remediation of a contaminated site more difficult. The only design that can counteract the effects of the downward gradient the creates the vertical transport is one that includes source zone pumping.

Figure 9.15: Plume mass reduction compared for the gravel trench (orange), gravel trench with pipe (yellow), and gravel trench with pipe and source zone pumping (green) designs for the heterogeneous intact bedrock scenario.
Chapter 10

Heterogeneous Fractured Bedrock Scenario

A scenario where the bedrock is highly conductive (fractured) and the overlying spatial distribution of the material is heterogeneous is now considered. The calibrated properties are summarized below in Figure 10.1. The hydraulic conductivity of 0.8 m/d assigned to the fractured bedrock material is consistent with a site estimate. The properties of the alluvial material vary by two orders of magnitude with a maximum conductivity of 4 m/d in the clean sand and a minimum of 0.04 m/d for the clay/silt material. Figure 10.2 displays the distribution of the materials. The same T-Progs generated material set is used for the alluvium. Differences between the material set used for this chapter and the heterogeneous intact bedrock scenario in the last chapter is that the upper bedrock in the last chapter has been converted to a fractured bedrock layer and the horizontal anisotropy is not a calibration parameter. The calibrated hydraulic conductivity values are also different this chapter.
Figure 10.1: Calibrated material properties for the heterogeneous fractured bedrock scenario.

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Figure 10.2: Spatial distribution of materials for the heterogeneous fractured bedrock scenario.
10.1 Groundwater Flow Calibration

Since the scenario where the bedrock at the site is fractured is what is expected for the site, the groundwater calibration did not require adjusting any parameters other than the hydraulic conductivity. As can be seen in Figure 10.3, the observed heads are in good agreement with the calculated heads at the site. The error bars indicate that the calibration targets for the wells are all but three of the thirty four wells used to calibrate the model are met (Figure 10.3). In the area where the source zone is placed and where the hydraulic barrier will be simulated, the wells are also in good agreement (Figure 10.4). The head contours for the model imply that groundwater flow is going more towards the river than the stream for the conditions considered in this scenario. The range in computed heads is 3.142 m whereas the observed heads have a range of 3.47 m. The mean residual for the heads is -0.09 m and the mean absolute residual is 0.25 m, while the root mean squared error is 0.30 m. A normalized root mean squared error is calculated to be 0.086 for this scenario which is less than the 10% rule of thumb and acceptable. In Figure 10.5, there is a plot that shows the relationship between the computed and observed heads which seems to be in good agreement as well.
Figure 10.3: The groundwater head contour results and error bars for the calibration targets. The units of the scale are in meters and the length along the stream is 320 m for reference.
Figure 10.4: The groundwater head contours and calibration error bars zoomed into the area of greatest importance for the heterogeneous fractured bedrock scenario. The unit for head is meters.
Upon simulation of the transport of a conservative tracer, the results confirm that flow is largely towards the river. The inclusion of a highly conductive layer above the bedrock and below the saprolite results in an enhanced downward component of flow and transport. The tracer was not present in the fractured bedrock initially and after 30 years of simulation, it is present in high concentrations. This effect is consistent with the simulation results obtained in Chapter 8, though, now there is a greater degree of heterogeneity in the aquifer model.
Figure 10.6: Concentration distribution after 30 years of transport for the base case heterogeneous fractured bedrock scenario. The units for concentration are in ppb and the length along the stream is 320 m for scale.

10.2 Gravel Trench Results

Figure 10.6 shows the results of a 60 year contaminant transport simulation with the inclusion of a gravel trench. Some degree of source zone is achieved, although there is still a noteworthy amount of transport occurring within the saprolite and fractured bedrock materials. Figure 10.8 shows that including a gravel trench as a hydraulic barrier resulted in a reduction plume mass reduction from 32.2 kg to 1.31
kg after 60 years of simulation, a reduction of 96%. Since the layer representing the fractured bedrock has a higher effective hydraulic conductivity than the saprolite and alluvium overlying it, most of the transport is taking place there. The effect of the hydraulic barrier is felt in the fractured bedrock layer and there is some hydraulic isolation taking place in the fractured bedrock layer, but this effect is limited.

![Figure 10.7: Concentration distribution after 60 years upon inclusion of the gravel trench (K=853 m/d) for the heterogeneous fractured bedrock scenario. The units for concentration are in ppb and the length along the stream is 320 m for scale.](image)

- (a) Layer 6 (alluvium)
- (b) Layer 8 (saprolite)
- (c) Layer 9 (fractured bedrock)
- (d) i=460 cross section (Z magnification=3)
Figure 10.8: Plume mass as a function of time for the case where no hydraulic barrier is installed (blue) and a case where inclusion of a gravel trench (K=853 m/d) occurs after 30 years (orange).

10.3 Gravel Trench with Pipe Results

The simulation results after 60 years, are shown in Figure 10.9 for a hydraulic barrier design that includes addition of pipe to the gravel trench (effective hydraulic conductivity of 8530 m/d). A visual comparison of Figures 10.7a and 10.7b with Figures 10.9a and 10.9b confirm that there is less tracer in the alluvium and saprolite for a design that includes pipes in the gravel trench. The slightly improved hydraulic isolation design leads to a reduction in plume mass from 32.2 to 1.05 kg after 60 years of simulation, a reduction of 97%. The resulting plume mass is 20% lower than the final plume mass obtained for a gravel trench design. Although the plume mass does not seem to differ much from the inclusion of a gravel trench design, the reduction of plume mass is noticeable in Figure 10.9.
Figure 10.9: Contaminant distribution after 60 years for the gravel trench with pipe (K=8,530 m/d) design for the heterogeneous fractured bedrock scenario. The units for concentration are in ppb and the length along the stream is 320 m for scale.
Figure 10.10: Plume mass as a function of time for the case where no hydraulic barrier is installed (blue) and a case where inclusion of a gravel trench with pipe (K=8,530 m/d) occurs after 30 years (orange).
10.4 Gravel Trench with Pipe and Source Zone Pumping Results

The results for a design that incorporates a source zone pumping well extracting at a rate of 400 L/d is shown in Figure 10.11. Source zone pumping is now included to the gravel trench with pipe design and the plume mass is reduced even further from 32.2 kg to 0.64 kg (Figure 10.12), a 98% reduction. Even at a low flow rate of 400 L/d, the inward and upward gradient created from source zone pumping reduces the plume mass even further. Despite the well being screened only in the saprolite layer, the effect of source zone pumping is felt throughout the aquifer and particularly in the fractured bedrock where it has reversed the vertical transport to the fractured bedrock. This scenario appears to reach steady state after 85 years.
Figure 10.11: Concentration distribution after 60 years upon inclusion of the gravel trench with pipe (K=8530 m/d) and a low flow source zone pumping well for the heterogeneous fractured bedrock scenario. The units for concentration are in ppb and the length along the stream is 320 m for scale.
Figure 10.12: Plume mass as a function of time without the inclusion of a hydraulic barrier (blue) compared to a case where the gravel trench with pipe and source zone pumping (orange) is included after 30 years for the heterogeneous fractured bedrock scenario.
10.5 Comparison

When plume mass calculations for the hydraulic source zone isolation designs are considered after 60 years of simulation, it is clear that each design is highly effective. The plume mass reduction that results from each design is plotted in Figure 10.13. A reduction of 96%, 97%, and 98%, is achieved for the gravel trench, gravel trench with pipe, and gravel trench with pipe and pumping respectively. Source zone pumping is less desirable than a completely passive design due to potential costs in energy and for treatment of contaminated water, but it is likely to be necessary for a scenario where the bedrock is highly fractured.

![Figure 10.13: Plume mass as a function of time compared for the gravel trench (blue), gravel trench with pipe (green), and gravel trench with pipe and source zone pumping (yellow) designs for the heterogeneous fractured bedrock scenario.](image-url)
Chapter 11

Conclusions

11.1 Summary

Hydraulic source zone isolation shows promise from the results presented in this research. The results obtained in Chapters 3 and 4 show how robust passive hydraulic source zone isolation designs are and that site heterogeneity may make passive hydraulic source zone isolation designs an even more attractive option than active source zone isolation designs.

Plume mass plots are used in this analysis to quantify the performance of hydraulic barrier designs. Plume mass is a representative quantity that is easily calculated in numerical models and is plotted over time. Though, the model boundary conditions, distance to boundaries, and plume trajectory must be accurate otherwise the results could be misleading. In the field however, hydraulic source zone isolation designs should be monitored and analyzed using mass discharge near the hydraulic barrier since mass discharge is more practical and easier to measure than plume mass.

The hydrogeologic scenarios considered in this work include an aquifer that overlies intact bedrock, an aquifer intersected by a highly permeable strata, and an
aquifer that overlies fractured bedrock. The ideal scenario is one where the hydraulic barrier is underlain by bedrock or material with low permeability. If this is the case then no source zone pumping is needed. For a scenario where the aquifer intersects a highly permeable bed, since the key indicator of performance is the conductivity contrast between the aquifer and the hydraulic barrier, source zone pumping is again not necessary as long as the contrast in conductivity is about 1000 or greater. When the bedrock is fractured, then the contrast in hydraulic conductivity between the aquifer and the hydraulic barrier is not enough to improve the performance of the hydraulic barrier. Source zone pumping may be necessary to prevent downward flow and transport through the fractured bedrock. As the extraction rate increases, the effectiveness of the system will as well in addition to the cost of operation and energy consumption. The fractured bedrock should be investigated prior to determining what extraction rate will be used in the source zone.

Two additional scenarios were also considered that incorporated a greater degree of heterogeneity and spatial variability in the aquifer. The first, simulated conditions similar to an aquifer overlying intact bedrock material. The performance of the hydraulic barrier in this circumstance was promising as well. In contrast to active hydraulic isolation designs, the notion that heterogeneity does not impair the performance of a hydraulic barrier and may in fact enhance from the conceptual model results is revisited. For a scenario where the bedrock is fractured, the limitations of increasing the effective hydraulic conductivity of the barrier resurface. Despite significant reductions in plume mass, source zone pumping may be the only option to noticeably improve the performance of a hydraulic isolation design that is underlain by fractured bedrock or analogously does not fully penetrate an aquifer.
11.2 Recommendations

Sites should be characterized as well as possible in order to design an optimal hydraulic isolation system. The hydraulic conductivity of the aquifer, depth to bedrock or confining unit, and characteristics of the bedrock or confining material are essential to determining whether or a passive hydraulic isolation design could be beneficial to the remediation goals of a project. Additionally, prior to designing a hydraulic isolation system, the contaminant source zone should be delineated as well as possible in order to determine the location and dimensions of a hydraulic barrier. The presence of any other plumes or sources near the upstream dimensions of a hydraulic barrier should be investigated as well. Figure 3.4b depicts the capture zone that a hydraulic barrier has on groundwater flow. This capture zone effect will enhance the downstream transport of any upstream contaminants present in the aquifer and it is important to investigate if any upstream contaminants are present within the anticipated capture zone. Monitoring wells will likely be installed in the hydraulic barrier to monitor the reduction in the head gradient through the source zone, which could be used as an indicator of clogging. The installed wells can also be used as extraction and injection wells in order to provide a preventative measure if clogging in the hydraulic barrier.

In addition to the data that was used in this study, the following may be necessary for the development of a more site specific model and the proposal of an optimal design:

- Seasonal measurements of the water level along the rio Paraiba so that an average can be computed for the steady state groundwater model boundary

- Seasonal measurements of the water level along the Goiabal so that an average can be computed for the steady state groundwater model boundary and a more
accurate estimate of conductance can be obtained

- Identifying the source zone geometry and vertical extent so that the optimal placement and geometry of the hydraulic barrier can be determined

- Delineating the plume for contaminant transport model calibration as well as computing the initial plume mass

- A cost estimate of construction of a trench with pipes so that cost can be considered in the selection of an optimal design

- Defining specific remediation goals for the hydraulic source zone isolation design to achieve

11.3 Future Work

This study does not consider the effects of clogging on the performance of a hydraulic barrier. Numerical models that can account for the clogging of the pores or pipe perforations in some of the proposed designs and assess the affects of clogging on the performance of a hydraulic barrier would be interesting. The simplification of using an effective hydraulic conductivity for a gravel trench with pipe could be relaxed and more detailed hydraulic isolation designs considered if an unstructured grid is used. MODFLOW-USG does provide a tool to do so and it does have a package that can account for the presence of pipes in in the subsurface with the Connected Linear Networks (CLN) package. However, MODFLOW-USG has not been coupled with the contaminant transport code MT3DMS during the time this study was being conducted. More detailed contaminant transport simulations can be done as well that deal with a non ideal source zone and account for processes such as sorption
and biodegradation. Also, the development of designs that use permeable reactive barriers not only to treat a plume but to hydraulically isolate source zones as well. The mass discharge out of a source zone would not only be reduced, it would also be treated when the contaminant comes into contact with the reactive hydraulic barrier. A field implementation of a passive hydraulic source zone isolation design should be considered and its performance closely monitored. Such a demonstration is the only way to assess whether or not this technology would work under the uncertainty inherent in the natural environment.
Appendices
A.1 Parallel Flow Analytical and Numerical Comparison

The effective hydraulic conductivity is given by a weighted average using Darcy’s Law and the Darcy Weisbach equation:

\[
K_{eff} = \frac{nD^4g\pi}{128\gamma} + \frac{KA_{trench}}{A_{total}}
\]

A 2D, horizontal model is used to simulate parallel flow in pipe and porous media and compare the results to the calculated effective hydraulic conductivity. Three pipes, 10 cm diameter pipes are installed in a gravel trench. The 2D model has a length of 10 meters and a height of 4.4 meters. The thickness used to compute the cross sectional area will be taken to be 1 meter. A hydraulic conductivity of 0.01 m/s is assigned to the gravel. The model geometry and dimensions are shown in Figure 1. The model boundaries are such that the pressures are kept constant at the inlet and outlet and create a head gradient across the model of 0.0005. No flow boundary conditions are assigned to the top and bottom boundaries of the model. Averages are taken along the outlets of model for the Darcy velocity and the Laminar flow velocity. The Laminar flow velocity is multiplied by the cross sectional area of the pipes while the Darcy velocity is multiplied by the cross sectional area of the trench minus the cross sectional area of the pipes to get the volumetric flow rate through the pipe and the porous media. The volumetric flow rate through the pipes and the gravel trench are summed and divided by the product of the total area of the hydraulic barrier and the head gradient.

The average laminar velocity out of the pipe is 0.86 m/s and the average Darcy velocity (which is uniform) is 5.10e-6 m/s. If both are multiplied by the appropriate
Figure 1: The dimensions of the 2D trench are 4.4 meters in width and 10 meters in length. Three pipes (smaller rectangles) are included in this trench and each pipe has a diameter of 10 centimeters.

cross sectional areas and summed, then a total volumetric flow rate of 0.02 m$^3$/s is obtained. Dividing the total volumetric flow rate by the total area of the trench (4.4 m$^2$) and the hydraulic gradient (0.0005), an effective hydraulic conductivity of 9.1 m/s is obtained. Using Equation (2.8), an effective hydraulic conductivity of 18.38 m/s is calculated. Although, a low head gradient is used, the Reynolds number in the pipe is $1.2 \times 10^5$, which is very high, the estimate is still within a factor of two. This serves as an example that equation 2.8 can provide a useful estimate of effective hydraulic conductivity.
Figure 2: Numerical simulation of parallel flow in pipe and porous media. The scale has units of m/s.
Bibliography


