8-2010

Hydromechanical Interference Slug Tests in a Fractured Biotite Gneiss

Trever Slack
Clemson University, tslack@blecorp.com

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ABSTRACT

Fractures are ubiquitous in the shallow crust and they commonly control the flow and storage of fluids in rock. Estimating transmission and storage properties resulting from fractures is commonly accomplished by interpreting the pressure signals caused by stressing an aquifer during a pumping or slug test. Fractures deform in response to pressure changes during well tests, and measuring and interpreting the deformation along with the pressure change is a way to potentially increase the information about storage and transmission properties. Tests where the pressure and deformation are coupled are called hydromechanical well tests. Previous investigations have focused on the effects of hydromechanical slug tests that use a single well. The single well slug test approach has the advantage of simplicity because it only requires one well, but it is limited in resolution compared to tests using multiple wells. The objective of this research is to improve understanding of fractured rock aquifers by including responses in monitoring wells and by integrating other data, such as borehole camera surveys, into well tests. The approach is to first characterize the response of a slug test using one source and one monitoring well by conducting theoretical analyses and field tests. The investigation shows that when the pressure is increased in the source well, the pressure change commonly drops in a monitoring well before it increases to give the expected signal. This reverse-water-level change in the monitoring well differs fundamentally from a similar response (the Noordbergum and Rhade effects) observed during pumping tests in confined aquifers because it occurs in the
same fracture stressed by the well, whereas those other classical effects occur in confining units overlying aquifers.

The investigation is expanded by conducting multiple tests using different combinations of wells to determine permeable paths formed by fractures. A simple search algorithm is used to first locate connections between long permeable intervals in boreholes, and then the lengths of the intervals are progressively reduced to refine the resolution. Straddle packers are used to isolate individual fractures identified using camera surveys in order to refine the resolution even further. The result is the 3-D permeable network created by the fractures in the vicinity of the wells.
ACKNOWLEDGMENTS

I am grateful to a number of people for their contributions, without which, this project would not have been possible. First, I would like to thank my advisor, and committee chair, Dr. Larry Murdoch for his insight, advice, and encouragement. His devotion to research and to his students is unparalleled, and my appreciation for the extensive time and effort that he dedicated to my graduate research is inexpressible. I would also like to thank committee members Dr. Ronald Falta and Dr. Stephen Moysey for their thorough review of my work and insightful comments.

I am indebted to my friend and colleague Dave Hisz, for his assistance during those long days in the field and countless hours in the lab, often to the detriment of his own project research. I would also like to thank Chris Willis, a superb engineer and Clemson alumni, who was crucial in the development and design of equipment vital to my research.

Although I officially started this project in 2006, I consider my years spent at Georgia Southern University under the guidance of Dr. Charles Trupe to have been a crucial time in my life that established the basis for all of my future geological endeavors. I would like to thank Dr. Trupe for his commitment to not just instructing, but actually teaching and imparting knowledge. His encouragement and advice, even now, are greatly appreciated.

Finally, I would like to express my appreciation for my family. This work would not have been possible without the love and support of my wife, Abby, who has been
there to encourage me from the very beginning. Lastly, I would like to thank my parents for their unconditional love and for all of the sacrifices that they have made to make me successful.
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wellbores are coded to symbolize varying degrees
of $K$ and $S_s$ as follows: low $K < 3.0e-6$ m/sec (dotted vector), moderate $K > 3.0e-6$ m/sec but $< 6.0e-6$
m/sec (dashed vector), high $K > 6.0e-6$ m/sec (solid vector); low $S_s < 6.0e-6$ sec$^{-1}$ (purple vector),
moderate $S_s > 6.0e-6$ sec$^{-1}$ but $< 3.0e-5$ sec$^{-1}$ (blue vector), and high $S_s > 3.0e-5$ sec$^{-1}$ (red vector). |
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Relative $K$ and $S_s$ values along connections between LAR-4 and LAR-3. Background colors correspond to low (red), moderate (yellow), and high (green) relative conductivity connections between wellbores identified during the MLS method. Vectors showing the connections between wellbores are coded to symbolize varying degrees of $K$ and $S_s$ as follows: low $K < 3.0e-6$ m/sec (dotted vector), moderate $K > 3.0e-6$ m/sec but $< 6.0e-6$ m/sec (dashed vector), high $K > 6.0e-6$ m/sec (solid vector); low $S_s < 6.0e-6$ sec$^{-1}$ (purple vector), moderate $S_s > 6.0e-6$ sec$^{-1}$ but $< 3.0e-5$ sec$^{-1}$ (blue vector), and high $S_s > 3.0e-5$ sec$^{-1}$ (red vector). .......................................................... 133

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Hydraulically dominant fractures or fracture zones identified at the Clemson University lower well field based upon interpretation of straddle packer and MLS method data.......................................................... 137
CHAPTER ONE
BACKGROUND

The flow of fluids through fractured rock is a process that has important implications in many disciplines ranging from groundwater remediation to oil recovery [Pyrak-Nolte and Cook 1988]. These fields require understanding of how fluids flow through fractures and fracture networks. In order to understand the complexity of fracture flow, it is first necessary to understand how fluids move through the primary unit of these networks, the individual fracture. However, it is not enough to only understand the flow of fluids through fractures, it is also important to understand the geologic characteristics, in-situ stresses, and mechanical properties that influence flow.

1.1 REGIONAL GEOLOGY

The study area for this project is located in the Piedmont physiographic region, which occurs along the eastern United States from Alabama northeast to Pennsylvania and New Jersey. It is bounded on the northwest by the Blue Ridge Mountains and on the southeast by the Atlantic Coastal Plain [Swain 1989] (Figure 1.1).

The geology of the Piedmont is marked by regolith overlying bedrock comprised of metamorphic and igneous rock [Daniel and Dahlen 2002; Gonthier 2009]. The regolith consists primarily of saprolite, which is derived from in place weathering of bedrock, and lesser amounts of alluvium and organic material [Overstreet and Bell 1965; Harned and Daniel 1989; Wenner and Dowd 2005]. The composition of the regolith is a mixture of clay materials, such as kaolinite and vermiculite, along with quartz, mica and
iron oxides [Heath 1989; Daniel and Dahlen 2002; White et al. 2002]. The bedrock of the Piedmont is primarily comprised of hornblende gneiss, granitic gneiss, biotite gneiss, mica-schists, and amphibolites [Miller 1990; Daniel et al. 1993].

Figure 1.1 Piedmont physiographic region extending from Alabama northeast to Pennsylvania and New Jersey [Swain 1989].

Sheet fractures are sub-horizontal openings, and they are observed in rock outcrops and quarries throughout the Piedmont [Williams and Burton 2005] (Figure
1.2). These types of fractures typically occur in the upper 100 m of the subsurface and this suggests that their formation is related to the changes in stress state during unloading by erosion [Holzhausen 1989; Martel 2004]. Sheet fractures are dilational, however, and even though the confining stress is reduced during unloading, it remains in net compression. As a result, unloading alone appears to be insufficient to explain the formation of dilational sheet fractures [Ramani and Twidale 1999]. The mechanism of formation of sheet fractures remains unclear, and there may be multiple mechanisms, so for the purposes of this thesis the term sheet fracture will be used as a purely descriptive term that means a horizontal to gently dipping crack.

Figure 1.2 Sheet fracture observed in the Fletcher Quarry in northern Rockdale County, Georgia [Williams and Burton 2005].

Sheet fractures have the capacity for storing and transporting important quantities of water [Wyrick and Borchers 1981; Williams and Burton 2005]. In many
areas, steeply dipping joints enable vertical flow of water between regolith and sheet fractures in the underlying bedrock. The steeply dipping fractures are also important components in forming interconnected fracture networks [Heath 1989; LeGrand 1989; Daniel 1996; Williams 2003].

1.2  FRACTURE CHARACTERIZATION

Fractures significantly influence the hydraulic properties of rock, especially in low permeability formations. Fracture spacing, interconnection between fractures, fracture surface roughness, aperture size, and orientation determine the porosity and permeability of rock masses, therefore considerable effort has been put forth to characterize their hydrologic and geometric properties [Noorishad and Ayatollahi 1982; Pyrak-Nolte et al. 1987; Muralidhar 1989; Barton and Stephansson 1990; Aydin 1991; Myer et al. 1995; Olsson and Gale 1995; Hsieh and Shapiro 1993; National Research Council 1996; Singhal and Gupta 1999; Faybishenko et al. 2000; Rutqvist and Stephansson 2003].

1.2.1  Idealized model of a fracture

Fractures can be characterized as broad, sheet-like features with in-plane dimensions much larger than their aperture [Cook 2003]. The aperture of fractures can vary from a few microns to several millimeters, whereas the in-plane dimension can be hundreds of meters [Snow 1968a; Snow 1968b; van Golf-Racht 1982; Segall and Pollard 1987].
The classical view of a rock fracture is a pair of smooth, parallel plates [Baker 1955; Huitt 1956; Snow 1968a; Snow 1970; Gale 1975; Gale 1977; de Marsily 1986]. A more realistic idealization represents a fracture as two opposing rough surfaces in partial contact with each other at asperities and separated in other areas to form void spaces (Figure 1.3) [Neuzil and Tracy 1981; Pollard and Aydin 1988; Dyke 1996; Liu et al. 2000; Indraratna and Ranjith 2001]. The magnitude of separation between the opposing surfaces is the fracture aperture [National Research Council 1996].

Figure 1.3 Idealized model of a fracture [modified from Indraratna and Ranjith 2001; Lomize 1951; Dyke 1996]

Stress on contacting asperities and fluid pressure in the voids hold the fracture open [Mourzenko et al. 1997; Pyrak-Nolte and Morris 2000; Murdoch and Germanovich 2006]. Changes in fluid pressure can alter the stresses acting on the asperities, which in turn causes the asperities to deform and the aperture to change [Pyrak-Nolte and...
Morris 2000]. Changes in aperture in response to pressure changes depend on the spatial distribution and fraction of contact area over a fracture surface and the fracture geometry [Hopkins and Cook 1990; Zimmerman et al. 1990].

Aperture is highly variable over the surface of many fractures and it influences both the hydraulic and mechanical properties [Tsang and Witherspoon 1981; Brown et al. 1986; Brown 1987; Pyrak-Nolte et al. 1987]. Multiple techniques have been utilized in the aperture characterization process, including linear profilometry [Swan 1981; Brown and Scholtz 1985; Gentier and Ries 1990; Roberds et al. 1990], casting techniques [Pyrak-Nolte et al. 1987; Hakami and Barton 1990], injecting translucent epoxy resins into fractures [Gale 1987; Gale 1990; Gentier et al. 1989; Billaux and Gentier 1990], nuclear magnetic resonance imaging [Kumar et al. 1997; Dijk and Berkowitz 1999], and X-ray computed tomography [Johns et al. 1993; Keller et al. 1999; Bertels et al. 2001].

1.2.2 Properties of a fracture

The voids along a fracture can create an interconnected network, and the geometric properties of this network control flow and transport within the fracture. The geometry of the void spaces is affected by the geological origin of the fracture, changes in stress after the fracture forms, mineral precipitation and dissolution as fluid flows through the fracture, chemical reactions with the wall rock, or through human actions such as withdrawal or injection of fluids [Nelson 1985; Laubach 1988; Kulander et al. 1979; Morrow et al. 1990; National Research Council 1996].
**Storativity**

The pore spaces within a fracture provide voids where water can be stored, and deformation of the voids changes the storage volume. Storativity, $S$, is defined in the context of a fracture as the volume of water released from storage per fracture area per decrease in hydraulic head [Bear 1979]

$$S = \frac{\Delta V}{A_{FRX}\Delta h}$$

where $V$ is the volume of water released from storage, $A_{FRX}$ is the area of the fracture, and $\Delta h$ is the drop in head within the void spaces. The storativity of a saturated fracture can be represented as [Doe et al. 1982]

$$S = \rho g\left(\frac{1}{k_n} + C_f \delta\right)$$

where $\rho$ is the density of water, $g$ is acceleration due to gravity, $k_n$ is the fracture normal stiffness defined as the ratio of the effective normal stress versus fracture aperture change [Zangerl et al. 2008], $C_f$ is the compressibility of water, and $\delta$ is the fracture aperture.

**Transmissivity**

The transmissivity of a fracture is related to the hydraulic aperture as [Doe et al. 1982; Witherspoon et al. 1980]

$$T = \frac{\delta^3 p_f g}{12 \mu}$$

where $p_f$ is the hydraulic pressure, and $\mu$ is the viscosity of the fluid.
where $\mu$ is the dynamic viscosity, $\delta_h$ is the hydraulic aperture of an open slot with parallel sides where the head loss per unit flow is the same as the real fracture. Variations in the size and connectivity of the void spaces can create a wide range of transmissivities [Berkowitz 2002]. In cases where minerals precipitate in fractures or where dissolution has occurred, the hydraulic aperture can be considerably less or greater than the aperture defined by mechanical displacement [Berkowitz 2002].

The connectivity of fractures is a critical feature controlling fluid movement in subsurface systems [Berkowitz 2002]. Fracture networks provide pathways for water movement and can have a much higher transmissivity than the enveloping rock matrix, so it is important to be able to characterize the networks in order to understand the behavior of a particular aquifer formation [Berkowitz 2002].

### 1.2.3 Fracture networks

Fracture networks can have an important influence on both storage and fluid flow rates [Bear et al. 1993; Sahimi 1993; Berkowitz 1994; Berkowitz and Adler 1998]. Characterizing a fractured rock aquifer not only requires understanding the properties of individual fractures, but also identifying the connection of hydraulically significant fractures within the rock matrix [Adler 1999]. Conductive fracture networks can include a large number of interconnected fractures or may be limited to a very small portion of the total fractures in the rock mass [National Research Council 1996].
Naturally occurring fractures typically intersect other fractures to create a network [Kehrman 1978]. These intersections may link one fracture with a more permeable neighbor, or with a fracture filled with material that restricts flow. Flow within a discrete network of fractures is strongly influenced by the size, connectivity, shape and geometry of the individual fractures [Neuzil and Tracy 1981; Long and Witherspoon 1985].

1.3 SLUG TESTS

Slug testing is a field technique developed as a simple means of estimating hydraulic conductivity in saturated materials [Hvorslev 1951]. These tests are often preferred over pumping tests because they require more modest equipment and less time, and are therefore less expensive [Hall and Chen 1996]. The slug test can be an important tool for obtaining in-situ estimates of aquifer properties in formations of low hydraulic conductivity where it may be impractical to conduct pumping tests [Butler 1997]. Additionally, slug tests do not require the removal of water during testing and are therefore beneficial at sites where disposal of contaminated water is an issue [Boulding 1993]. Slug tests have been routinely utilized in hydraulic site characterizations, remedial investigations, and oil well performance tests [Shapiro and Hseigh 1998; Rutqvist 1995].
1.3.1 Conventional slug test

The basic slug test procedure consists of creating a rapid change in hydraulic head in a borehole. This causes water to flow from the well-bore into the formation as the head in the well returns to static conditions. The changes in hydraulic head are recorded as a function of time and can be analyzed to estimate hydraulic conductivity and specific storage of the aquifer formation [Butler 1997]. Conducting a conventional slug test requires the application of a method for changing the hydraulic head in the well-bore. Commonly used methods for creating these head changes include (1) removing a known volume of water using a bailer, (2) adding a known volume of water into a well, or (3) using a weight to displace a volume of water in a well [Butler 1997]. Although these methods are applicable in many groundwater investigations, their suitability for use in areas where contaminated ground water is suspected is limited when there is a possibility that the water may pose a serious health risk [Butler 1997].

1.3.2 Air-slug test

Leap [1984] developed an alternative to conventional slug testing methods by utilizing pressurized air to change the head in a well. These so-called air-slug tests are conducted by pressurizing the air inside the well casing and monitoring the change in water pressure as the air-water interface in the well drops and approaches equilibrium. The air pressure is then released and the rising pressure is monitored as it returns to initial levels [Air Slug version 1.1 user manual 1996; Shapiro and Greene 1995].
Air-slug tests provide a means of estimating the transmissivity and storativity of aquifers, and are preferred to conventional slug testing methods because they (1) can be designed to accommodate any diameter well-bore, (2) are repeatable, and (3) allow for larger slug volumes than traditional methods, which in turn affects larger volumes of fractured rock [Svenson 2006]. In addition, water does not need to be handled during air-slug tests, which simplifies logistics at locations of suspected groundwater contamination [Butler 1997].

The equipment needed to conduct an air-slug test is constructed and assembled at the top of the well casing. The pressurization of the well can be controlled using a wellhead assembly with valves that can either allow pressurized air to flow into the casing or vent the pressurized air to the atmosphere [Svenson et al. 2005] (Figure 1.4). The only equipment in contact with the water is a submersible down-hole pressure transducer used to measure pressure transient data [Air Slug version 1.1 user manual 1996].
1.3.3 Slug test analysis

The pressure transient signal of a slug test is characterized by a rapid increase in pressure followed by a gradual decrease that can resemble a negative exponential function of time as the water flows into the formation. The rate of pressure decline varies depending on the aquifer transmissivity. In addition, factors such as wellbore skin, interactions with other fractures, and heterogeneities may affect the pressure signal during a slug test [Ramey and Agarwal 1972; Doe and Osnes 1985; Karasaki et al. 1988; Shapiro and Hsieh 1998].

Methods for analyzing slug test data were developed as manual curve-fitting techniques based upon graphical procedures [Butler 1997]. The analytical solution for a water level drop in a finite diameter well after an instantaneous surge of water in a confined aquifer was first given by Cooper et al. [1967] and modified by Papadopulus et al. [1973] and included the parameters...
\[ \alpha = \frac{r_w^2 S}{r_c^2} \]  \hspace{0.5cm} (1.4)

and

\[ \beta = \frac{Tt}{r_c^2} \]  \hspace{0.5cm} (1.5)

where \( r_w \) is the effective radius of the well, \( r_c \) is the radius of the casing, \( S \) and \( T \) are the storativity and transmissivity, and \( t \) is time (Figure 1.5) [Batu 1998; Park and Zhan 2003].

![Figure 1.5](image)

**Figure 1.5** Dimensionless head as a function of dimensionless time during slug tests with different values of \( \alpha \) from Cooper et al. [1967]
1.4 INTEREERENCE SLUG TESTS

Slug tests are commonly conducted using a single well and their response is interpreted to reflect properties in close proximity to the well [Ferris and Knowles 1963]. However, including data from monitoring wells can increase the volume of the formation investigated by slug tests. Novakowski [1989] presents results from a monitoring well 15 m away from the test well, whereas Spane et al. [1996] show response data from monitoring wells located at 9 m and 11 m away from a well in an unconsolidated sand and gravel. The response in observation wells is expected to reflect formation properties between the test well and the monitoring well (Williams and Paillet 2002).

These field studies are consistent with theoretical results indicating the scale of distances that can be affected by slug tests. Ramey et al. [1975] showed that pressure responses should be measurable at distances up to several hundred times the effective screen radius of the test well (several tens of meters for wells 0.15 m in diameter).

Interference slug tests require two or more wells, where the slug is applied at one well and the response is measured at one or more observation wells. To reduce storage effects that limit the interference response, the fluid column of the observation well is often isolated using a packer system [Spane et al. 1996; Butler 1997].

Straddle packer systems can also be used to isolate short intervals of the test well. Hydraulic well tests with straddle packers can be repeated at multiple depths to create profiles of aquifer properties along wellbores [Davison and Keys, 1988] (Figure
1.6). These profiles are particularly useful in fractured rock aquifers where $T$ and $S$ can vary sharply with variations in the geometry of the fracture network [Hsieh and Shapiro 1993; Zlotnik and McGuire 1998; Lewis et al. 2000; Karasaki et al. 2000; Stephenson et. al. 2005; Ross and McElwee 2007].

Figure 1.6 Application of straddle packers system to isolate short fracture intervals. Hydraulic tests can be repeated at multiple depths (B) to create profiles of aquifer properties along a well-bore.
The typical response of a monitoring well is characterized by an initial rise in pressure followed by a flat transitional segment and then by a declining, recessional limb segment. According to Spane [1996] the shape and amplitude of the wave are primarily controlled by the storativity and anisotropy, whereas transmissivity is the principal parameter affecting the arrival time of the slug pulse.

1.5 HYDROMECHANICAL SLUG TESTS

The influence of deformation on aquifer tests can be studied by measuring in-situ changes in fracture aperture. Mechanical displacements in fractures have been measured in wells using borehole extensometers [Thompson and Kozak 1991; Hesler et al. 1990; Martin et al. 1990; Schweisinger and Murdoch 2002; Cornet et al. 2003; Murdoch et al. 2004; Cappa et al. 2005; Cappa et al. 2006]. Gale [1975] was the first to directly measure the hydromechanical response of fractures to well tests by using an extensometer to measure axial displacements in response to fluid pressure change.

1.5.1 Moveable borehole extensometer

A borehole extensometer is a device capable of measuring axial displacements along a borehole. A new extensometer design was developed to advance the resolution, mobility, and ease of deployment compared to existing devices [Murdoch et al. 2004]. Previous attempts to directly measure in-situ fracture displacement have made use of extensometers that were tedious to move, limiting the applicability of well
tests to specialized conditions. A modified extensometer that could be readily mobilized and is capable of resolving displacements on the order of approximately 0.15 μm was described by Murdoch et al. [2004] and Schweisinger et al. [2006].

The extensometer consists of two anchors separated by a split reference rod that is temporarily fixed together by a registration pin. The anchors consist of remotely actuated, pneumatic cylinders that push carbide grippers into the face of the borehole with several hundred pounds of force [Murdoch et al. 2004; Schweisinger 2007] (Figure 1.7).

![Figure 1.7](image_url)  
**Figure 1.7** Remotely actuated pneumatic cylinder used to anchor the extensometer into the borehole wall. The cone-shaped piece registers and locks the extensometer to its frame when the anchors are retracted [Schweisinger 2007].
The extensometer is disengaged from the frame during tests so it is unaffected by small movements of the frame and only responds to rock displacement. A registration pin is extended to fix together the two reference rods during deployment along the borehole. The pin is retracted during testing, so the rods and anchor points are independent of one another. A high-resolution linear variable differential transducer (LVDT) is used to measure the displacement of the anchor points with sub-micron resolution [Schweisinger et al. 2005] (Figure 1.8).

The rods connecting the anchors to the measuring device are made of invar, a material with an exceptionally low coefficient of thermal expansion (0.7x10^{-6} inch/(inch °F) in order to minimize the sensitivity of the device to temperature changes [Murdoch et al. 2004; Svenson 2006]. A complete description of the moveable borehole extensometer can be found in Schweisinger [2007].
As a result of extensive testing since 2001 at Clemson University, it has been shown that the moveable borehole extensometer can measure axial displacements with a resolution of approximately 0.15 micron during hydraulic well tests [Svenson et al. 2008; Svenson et al. 2007]. The displacement data can improve the interpretation of hydraulic tests in fractured rock aquifers when analyzed using hydromechanical analyses that couple mechanical displacement with fluid dynamics [Rutqvist et al. 1998; Rutqvist and Stephansson 2003; Cappa et al. 2005a; Cappa et al. 2005b; Cappa et al. 2006; Schweisinger et al. 2005; Murdoch and Germanovich 2006].
Displacement measurements can be evaluated by plotting them as functions of time or pressure head [Svenson et al. 2008] (Figure 1.9). These plots have distinctly different shapes for different formation properties. The magnitude and distribution of the displacement will depend on the distribution of aquifer parameters, suggesting that coupling pressure signals with displacement could improve the resolution of aquifer characterization [Svenson et al. 2008; Schweisinger et al. 2007].

![Figure 1.9](image-url)  
**Figure 1.9** Typical response of a hydromechanical well test. a) pressure head and displacement during a slug test as a function of time b) displacement during a slug test plotted as a function of pressure head [Schweisinger 2007].

Understanding the interaction between mechanical deformation and fluid flow in fractured rock is an important aspect in evaluating aquifer performance. However, evaluating the hydromechanical interactions within fractured rocks is often difficult, largely due to the complexity of the fracture network geometry and the heterogeneity in both the fracture and rock matrix hydromechanical properties [Cappa et al. 2005]. Coupled hydromechanical effects within one fracture depend not only on its hydraulic and mechanical properties, but also on the hydraulic and mechanical connections with
other fractures, the orientation and magnitude of effective stresses imposed on fracture walls, and orientation or dip of the fractures [Cappa et al. 2006; Svenson 2007].

1.5.2 Poroelasticity

The presence of fluid in porous rock has a major influence on the behavior of a rock mass [Jaeger et al. 2007]. The majority of work on subsurface flow problems in hydrogeology, petroleum engineering, and geophysics is conducted on the premise that the rock mass deforms volumetrically in response to local pressure changes. This effect is taken into account using the concept of specific storage in the diffusivity equation to analyze pressure transients [Bear 1979; Fetter 2001; Malama and Barash 2009]. This assumption is often viable for representing flow during some processes, but other situations require the coupling between mechanical deformation and pore fluid pressure to be evaluated in more detail [Jaeger et al. 2007].

The study of the relationship between elastic deformation and pore fluid pressure originated from the works of Terzhagi [1936] in describing the one dimensional consolidation of soils. The term *poroelasticity* was first coined by Geertsma [1966] in reference to Biot’s work [1941, 1956, 1973] on the theory of the elasticity and viscoelasticity of fluid-saturated solids. Poroelasticity is used to analyze stress changes during fluid extraction or injection in fluid-saturated rock formations [Settari 2002; Fabian and Kumpel 2003; Kim and Parizek 2005; Kim and Parizek 1997; Yin et al. 2007; Tseng et al. 2008] or how changes in loads applied to an aquifer change water-level in
wells [Wang 2000; Schevenels et al. 2004; Theodorakopoulos et al. 2004; Comerlati et al. 2004; Tsai 2009]. This analysis has been used in geomechanics, hydrogeology, and petroleum engineering [Dusseault 1999; Wang 2000; Zhang et al. 2003].

The two basic concepts underlying poroelasticity are solid-to-fluid coupling where a change in applied stress produces a change in fluid pressure or fluid mass, and fluid-to-solid coupling where a change in fluid pressure produces a change in the volume of the porous material [Wang 2000]. For fractures, changes in aperture can occur as a result of changes in applied stress, which in turn is accompanied by a fluctuation in fluid pressure. Increasing fluid pressure opens the aperture, which in turn affects the distribution of fluid pressure [Schweisinger 2007]. As a result, discrete fractures can be treated as thin layers that deform using the same coupling between fluid pressure and stress that is used in poroelasticity [Murdoch and Germanovich 2006]. The thin geometry of discrete fractures changes the definition of some of the terms compared to conventional poroelasticity, which is formulated for three-dimensions. For example, the Biot-Willis parameter, $\alpha$, [Biot and Willis 1957] has a slightly different meaning when applied to fractures [Murdoch and Germanovich 2006], but this is a minor difference that is readily accommodated.

The use of poroelastic analyses is important for this investigation because of the relationship between fracture displacement and changes in fluid pressure during slug tests. Conventional theories describing transient flow through porous media predict that deformation will only respond to the local pressure change, whereas observations
have shown that this is not true. Poroelastic analyses provide the means to accurately predict the coupling between deformation and pressure change, so this approach will be used to interpret the field data.
CHAPTER TWO
RESPONSE OF AN INTERFERENCE SLUG TEST

2.1 Overview

Slug tests are of particular importance for hydraulic characterization of contaminated sites where disposal of groundwater can be expensive. Conventional slug tests are commonly performed using a single well and are expected to affect a small volume in the vicinity of the well [Ferris and Knowles 1963; Ferris et al. 1962]. However, the analysis of additional signals from monitoring wells during interference slug tests can provide insight into parameters typically not available during single well tests [Spane 1996].

In its simplest form, an interference slug test uses a pair of wells, one acting as the source well and the other as a monitoring well [Black and Kipp 1977; Spane 1996]. The pressure response in the monitoring well can be analyzed along with the pressure signal in the stressed well to provide average values of the hydraulic properties of the formation between the well-bores [Novakowski 1989; He et al. 2006]. A straddle packer system with a submersible transducer can be used to measure the response along an interval in the monitoring well, and the system can be moved to isolate different locations along the borehole [Davison and Keys 1988; Spane et al. 1996; Butler 1997].
Interference slug tests were conducted in a fractured biotite gneiss at the Clemson University well field and the type curve in the monitoring well includes four characteristic stages:

Stage 1. An initial lag period between the time when pressure is increased in the source well and the time when the pressure changes in the monitoring well.

Stage 2. The pressure decreases, typically marking the initial response in the monitoring well.

Stage 3. The pressure increases and reaches a peak.

Stage 4. The pressure decreases to ambient conditions.

The appearance of a decrease in pressure during Stage 2 was surprising, and we are unaware of previous descriptions of this effect during interference slug tests. Currently available analyses of interference slug tests are unable to predict this response. Nevertheless, the Stage 2 response appears to be typical of many of the interference tests conducted at the Clemson University well field.

This effect is readily predicted using a theoretical analysis of a slug test that couples elastic deformation and fluid flow within a fracture. It appears that the response results from a poroelastic behavior that is overlooked by conventional slug test analyses. The poroelastic analysis provides an opportunity to analyze this type of response and therefore to potentially increase the information available from interference slug tests.
2.2 INTRODUCTION

The interference slug test method requires two or more wells and the test is conducted by rapidly raising or lowering the head in the source well in the same manner as a conventional slug test. The response is then measured at one or more observation wells [Black and Kipp 1977; Spane 1996; Butler 1997]. McElwee et al. [1995b] showed that the addition of a monitoring well during a slug test can greatly improve the accuracy of storativity values.

Conventional slug tests are typically expected to respond to aquifer properties in relatively close proximity to the well [Ferris and Knowles 1963; Rovey and Cherkauer 1995; Shapiro and Greene 1995]. However, field data from monitoring wells in interference slug tests have shown that the slug test can affect pressures up to 10 m or more. Novakowski [1989] presents field data from a monitoring well located 15 m away from the test well whereas Spane et al. [1996] shows data from monitoring wells located at 9 m and 11 m away from the test well in an unconsolidated sand and gravel. Pulido et al. [2004] were able to measure pressure responses at a monitoring well located 70 m away from the source well using large displacement slug tests with initial head displacements greater than 5 m. These data suggest that interference slug tests may be sensitive to formation properties up to several tens of m from the source well. Traditionally, initial head displacements during slug tests have been 1 m [McElwee and Zenner 1998] or lower. For example, Cooper et al. [1967] used initial displacements of 42 cm in their slug tests.
The typical response at a monitoring well to a slug-in test is a pressure pulse characterized by an initial pressure increase followed by a period of gradual change, and then a declining, recessional limb segment [Spane 1996]. According to Spane [1996], the shape and amplitude of the pulse are primarily controlled by the elastic characteristics (storativity) and anisotropy, whereas the average transmissivity primarily affects the arrival time. A semi-log plot of the theoretical pressure response in a monitoring well located at a distance of 100 times the effective screen radius calculated using the Cooper et al. [1967] method show the theoretical effects of both transmissivity and storativity on the monitoring well response (Figure 2.1).

**Figure 2.1** Sensitivity analysis of transmissivity and storativity on the response of a monitoring well located at a distance of 100 times the effective test well radius with a 6 m head change at the test well.

Interference slug tests conducted in a fractured biotite gneiss in Clemson, South Carolina cause similar responses in monitoring wells, with the exception of an initial decrease in the pressure signal (Figure 2.2) herein referred to as a reverse water level
fluctuation (RWF)) prior to the pressure pulse arrival. To our knowledge, this response during interference slug tests has not been described in literature and appears to be a result of a poroelastic effect.

The relationship between changes in fluid pressure and changes in volume of porous material has long been observed in nature and described extensively in literature [King 1892; Pratt and Johnson 1926; Meinzer 1928; Jacob 1940; Biot 1941; Hughes and Cook 1953; Biot 1956; Hubbert and Willis 1957; Haimson and Fairhurst 1969; Geertsma 1966; Gambolati and Freeze 1973; Segall 1992; Roeloffs 1988; Detourney and Cheng 1993; Hart and Wang 1995]. A comprehensive discussion of the development of poroelastic theory with application to hydrogeology can be found in Wang [2000].

Field observations of an initial rise in hydraulic head occurring in aquifers and aquitards adjacent to a pumped aquifer during the early times after the pump is turned on were reported as early as 1936 by Barksdale et al. Similar pressure responses were

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**Figure 2.2** Example of initial decrease in the pressure signal at a monitoring well located 5.3 m from a test well during an interference slug test conducted in a fractured biotite gneiss in Clemson, South Carolina.
reported during pumping tests by Ferris et al. [1962], Andreasen and Brookhart [1963], and Van Eyden et al. [1964].

It was not until 1969 that Verruitt coined the term “Noordbergum effect” to describe a rise in the water level in monitoring wells located near a pumping well. The anomalous water level rise occurred for almost an hour after the pump was turned on during a test conducted near the town of Noordbergum in the Netherlands. Verruitt explained the reverse water level fluctuation as occurring because pumping compressed the aquifer to force water up into the monitoring wells [Yelderman 1983]. The opposite response of the Noordbergum effect, which consists of an initial drop in hydraulic head of nearby monitoring wells when the pump has been turned off, was termed the “Rhade effect” [Languth and Treskatis 1989].

Other notable observations of the Noordbergum effect include Wolff [1970], Rodrigues [1983], Languth and Treskatis [1989], Broska et al. [1999], and Chen et al. [2005]. Modeling efforts to simulate the Noordbergum effect include Hsieh [1996] who developed a three layer model and finite-element code to simulate the Noordbergum effect during a pumping test, and Kim and Parizek [1997] who conducted numerical simulations of a layered aquifer to investigate the mechanisms behind the response. Kim and Parizek [1997] concluded that the Noordbergum effect was likely caused by two mechanisms; 1) a faster mechanical propagation (deformation) of the pumping stress than its hydraulic propagation (drawdown) from the pumped aquifer into the
adjacent aquitard and unpumped aquifer, and 2) a mechanical amplification (excessive compression) in the lower part of the relatively soft aquitard.

In general, the characteristic Noordbergum effect observed in the field occurs when groundwater is pumped from an aquifer and hydraulic heads in adjacent aquifers and aquitards increase almost immediately after the start of pumping and then eventually decline [Kim and Parizek 1997]. The magnitudes of the Noordbergum head rise are typically reported to be on the order of centimeters and can often disappear quickly. As a result, the head responses are often unable to be detected beyond background noise using conventional monitoring devices, or are disregarded in the analysis of aquifer pumping tests [Kim and Parizek 1997].

2.3 FIELD SETTING

The field site lies within the South Carolina Botanical Gardens located south of Perimeter road in Clemson, South Carolina (Figure 2.3). A cluster of wells was installed in an area south of the Bob Cambell Geology Museum by the Department of Geological Sciences at Clemson University in 1999 to characterize the local groundwater system.
Figure 2.3 Field site located within the South Carolina Botanical Gardens in Clemson, SC. A cluster of four bedrock wells serves as the primary experimental site for well testing development and fracture characterization research at Clemson University.

This well cluster serves as an experimental site for well testing development and fracture characterization research at Clemson University. The boreholes in the lower well field were installed using hollow stem auger and air-rotary techniques. LAR-2, LAR-3, and LAR-4 were drilled to 60 m depths, whereas LAR-1 was drilled to a depth of 120 m. Each was cased with 0.15 m diameter casing through approximately 21 m of regolith.
2.3.1 Geology

Core samples from LAR-2 show the dominant rock type as being medium-grained biotite gneiss with strong foliation that strikes northeast and dips between 40 and 80 degrees to the southeast [Svenson 2006]. The rock is composed predominantly of quartz, plagioclase, and biotite with lesser amounts of hornblende, epidote, garnet, and chlorite.

A borehole camera survey was used to locate visible fractures within the boreholes. Fracture density is 3 to 4 m⁻¹ at a depth of roughly 20 m in LAR-3 and LAR-4, but decreases with depth and is 0.1 to 0.4 m⁻¹ below 50 m. Fracture density is fairly uniform in LAR-2 down to 50 m, however no fractures were observed below that depth. In contrast, fracture density is 0.1 to 0.3 m⁻¹ in LAR-1, with an exception at 80 m where there are three relatively closely spaced fractures (Figure 2.4).
2.4 FIELD EXPERIMENTAL SETUP

A pneumatic air slug system was used to investigate the interference slug test response. The setup includes three vertical monitoring wells (LAR-3, LAR-2, and LAR-1) located at 5.3 m, 11.2 m, and 13.4 m respectively, away from a test well (LAR-4). A pair of inflatable packers was used in each well to isolate individual fractures or fracture sets and measure the hydraulic response from a slug test initiated in the test well.

Figure 2.4 Fracture locations identified during a borehole camera survey of the field site.
2.4.1 Air slug equipment

The pneumatic air-slug system consists of a well-head assembly, a 2-inch PVC pipe string extending down to a pair of straddle packers, and a down-hole pressure transducer. The pressurization of the casing was controlled using an assembly consisting of a regulator and a series of valves and pressure gauges. A three-way valve was used to pressurize the well from a pressure-regulated air source, or to vent the casing to the atmosphere. A submersible transducer was lowered below the water level in the casing, and the cable to the transducer extended through a compression seal in the well-head assembly.

2.4.2 Packer systems

The packers used in the test well during the interference slug tests were RocTest LP 102-190, with a rubber gland that contacts a length of the borehole spanning approximately 0.7 m. The isolated interval between the packers spanned 3 m from the top of the lower packer to the bottom of the upper packer. The packers were inflated with N₂ gas to approximately 1.2 MPa (180 psi) above hydrostatic pressure. Low-pressure packers [Svenson et al. 2005] were used in the monitoring wells and were inflated to approximately 0.2 MPa (29 psi) above hydrostatic pressure. The low-pressure packers contacted the borehole wall over approximately 0.4 m and were separated by a distance of 2.4 m.
2.4.3 Data Acquisition

The pressure responses from the slug tests were monitored using a pressure transducer in the test well and in each of the monitoring wells. A Wika Submersible Liquid Level Transmitter (Model LS-10) with a span of 50 psi was used to monitor pressure in the test well (LAR-4) and one with a 15 psi span was used in monitoring well LAR-3. Accuracy of the Wika transducers is approximately 1.0% of the full scale, according to the manufacturer. Pressure transducers in monitoring wells LAR-2 and LAR-1 were Honeywell Wet/Wet Differential Pressure Sensors (model PX26) with spans of 5 psi and 1 psi, respectively. The Honeywell transducers were configured with operational amplifiers and then encapsulated in epoxy for deployment downhole. Amplifying and calibrating the Honeywell transducers yielded accuracies of approximately 0.1% of the full scale. The transducers used a 0-5 VDC output, which was interfaced with a 24-bit analog-digital converter and the resulting signal was recorded by a National Instruments data acquisition system at a rate of 1 HZ.

2.4.4 Test Procedure

To conduct a slug test, the solenoid valve on the well head of the test well was opened to a tank of pressurized air attached to a pressure regulator. Air flowed into the head space above the water level inside the PVC pipe string and the pressure increased to the pre-set maximum pressure of the pressure regulator over a period of approximately 8 seconds. This caused an abrupt increase in pressure at the level of the
fracture, and is equivalent to dropping a weighted slug into the well during a conventional slug test. The water level in the PVC pipe string was lowered in response to the pressurized head space, and this was measured by the submersible transducer. Eventually, the water level in the pipe string dropped by an amount equal to the hydraulic head in the tank and the system was equilibrated. The solenoid valve was then vented to the atmosphere, which rapidly dropped the pressure inside the pipe string and initiated a slug-out test. The pressure dropped over 5-6 seconds during the slug-out test, several seconds faster than pressurization during the slug-in. The solenoid valve was attached to an electric timer so that slug tests could be repeated automatically.

2.5 FIELD RESULTS

Approximately 120 interference slug tests were conducted in the Clemson University lower well field during this phase of work. Previous testing [Svenson 2006; Schweisinger 2005] at the well field utilized primarily single well tests and identified three fracture zones; an upper zone (~25 m), a middle zone (~36 m), and a lower zone (~50 m) within the LAR-4 borehole that have a relatively high transmissivity ($T \approx 10^{-4} \text{ m}^2/\text{sec}$) (Figure 2.5). The higher transmissivity zones are separated by spans of borehole with a relatively low transmissivity ranging from approximately $10^{-5}$ to $10^{-6} \text{ m}^2/\text{sec}$ [Svenson 2006]. A majority of the interference slug tests performed during this work were concentrated on identifying and characterizing the interference response between
these highly transmissive fracture zones and monitoring wells LAR-3, LAR-2, and LAR-1.

Examples of the interference response observed in the monitoring wells from slug tests initiated in each of the higher transmissivity intervals in LAR-4 (designated Zone I, II, and III) are illustrated on Figures 2.6 through 2.8. The slug tests involved an initial head change of approximately 3 m occurring over an average 8 sec period. Each test is identified by 3 criteria:

1. Depth zone in LAR-4 that was stressed (Zone I, II or III)

2. Monitoring well (LAR-1, LAR-2, LAR-3)

3. Depth zone in the monitoring well (a, b, c, etc).

The tests will be designated by specifying the locations where the pressure originated and the interval where it was detected. For example, a test originating in Zone II in LAR-4 and detected at depth zone c in LAR-3 would be: LAR-4 Zone II→LAR-3c. This will be abbreviated as: 4II-3c.
The responses involve an increase and decrease in pressure that appears roughly symmetric on a semi-log plot (Figure 2.6). Maximum pressure head is 0.10 m in test 4III-3d, but most peak pressures are on the order of cm. The times when the peak pressures occur range from 10 s to several 100 s (Figures 2.6 through 2.8). The time scale starts when the valve is opened and the pressure begins to increase in the source well. Some signals (e.g. 4III-3d) appear noisier because the small pressure response is at the noise level of the transducer. The noise level during other tests with small signals (e.g. 4I-2a) is much lower because transducers with a shorter span were used to measure them. In general, the pulse-like responses during the tests with relatively large pressure signals resemble the type curve in Figure 2.1 and they can be analyzed by standard parameter estimation methods. This approach will be pursued in more detail in the following section.

Upon careful inspection, it is apparent that a RWL signal occurs at the beginning of many of the tests. For example, the pressure head decreases by approximately 0.01 m at the beginning of tests 4I-3a and 4II-3c and the trough of the RWL response during these tests occurs between t = 2 to 5 seconds. A RWL is absent from 4II-3b and 4III-3e, but this may be because the RWL occurred too quickly to be measured by the 1 Hz data logging rate. The response early in 4II-3d is ambiguous because of the low signal to noise ratio.
A RWL response is clear in all the pressure signals measured at LAR-2 (Figure 2.7) and is characterized by an amplitude between 0.008 m and 0.025 m. The trough of the RWL occurs at $t = 30$ to $40$ s in most cases, and the RWL persists for nearly $100$ s during these tests. The peak pressures during these tests are similar in magnitude to the RWL.
ranging from 0.01 m to 0.03 m. A striking exception occurs at the lowest depth interval (test 4III-2I), where the trough of the RWL occurs much more quickly ($t = 4$ s) and the total duration of the RWL is approximately 8 s.

Figure 2.7 Interference slug test pressure response in LAR-2 with depth.
Figure 2.8 Interference slug test pressure response at LAR-1 with depth.

The RWL during two of the tests measured at LAR-1 (4I-1a, 4II-1a) resemble the responses at the upper levels of LAR-2 (Figure 2.8). Two other responses measured at
LAR-1 are different, however, in that no RWL is evident. The first response is a pressure increase during tests 4II-1b and 4III-1c.

2.5.1 Repeatability

Multiple slug tests were performed at each test location to evaluate the repeatability of the interference pressure response. At each location, four tests were conducted with an initial slug pressure of 27.5 kPa, and they were followed by four more tests using an initial pressure of 55.1 kPa. The testing apparatus used a timer so each test was separated by approximately 30 minutes from the next one. The sets of four tests from monitoring wells LAR-2 and LAR-1 are essentially identical. Variations between tests are similar to the noise in the individual datasets. The scatter of data from LAR-3 is larger than from the other two monitoring wells, but this is because the measurements were made with a less accurate transducer calibrated to measure a larger span of pressure. The measurements are reproducible within the noise level of the sensor at LAR-3, just as at the other two monitoring wells (Figure 2.9). Standard deviations of the minimum pressure heads in the troughs of the RWL are a half to one third those of the peak positive pressure signals (Table 2.1).
Figure 2.9 Head as functions of time for four consecutive slug tests using 2 different air pressures at three different wells. White circles denote 27.5 kPa slug-in tests. Dark gray circles denote 55.1 kPa slug-in tests.
Table 2.1: Summary of Standard Deviations (m) for Multiple Slug Tests

<table>
<thead>
<tr>
<th>Monitoring Well</th>
<th>Test ID</th>
<th>Initial Slug Pressure (kPa)</th>
<th>RWL Trough Standard Deviation (m)</th>
<th>Peak Positive Pressure Standard Deviation (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MW-3</td>
<td>4I-3a</td>
<td>27.5</td>
<td>0.006</td>
<td>0.016</td>
</tr>
<tr>
<td>MW-3</td>
<td>4I-3a</td>
<td>55.1</td>
<td>0.008</td>
<td>0.011</td>
</tr>
<tr>
<td>MW-2</td>
<td>4I-2b</td>
<td>27.5</td>
<td>0.0018</td>
<td>0.0034</td>
</tr>
<tr>
<td>MW-2</td>
<td>4I-2b</td>
<td>55.1</td>
<td>0.0018</td>
<td>0.0035</td>
</tr>
<tr>
<td>MW-1</td>
<td>4I-1a</td>
<td>27.5</td>
<td>0.0009</td>
<td>0.0016</td>
</tr>
<tr>
<td>MW-1</td>
<td>4I-1a</td>
<td>55.1</td>
<td>0.0011</td>
<td>0.0025</td>
</tr>
</tbody>
</table>

2.5.2 Characteristic responses

Data sets in which the RWL signal is well expressed were chosen from each monitoring well to be used for further analysis throughout this thesis (Figure 2.10). In this particular dataset, the pressure head at LAR-3 (5.3 m from the test well) is static for approximately 2 seconds, and then drops by 0.008 m at 4 seconds. The signal then begins to increase and reaches a peak at 15 seconds before falling back to static conditions. The pressure responses in the other two monitoring wells (LAR-2 and LAR-1) are qualitatively similar, although the magnitude and time of the initial drop in pressure head are different, ranging from 0.010 m to 0.015 m in magnitude and occurring as long as 25 seconds after the start of the test.
Figure 2.10 Typical head response in the test well (LAR-4) and monitoring wells LAR-3 (5.3 m from the test well), LAR-2 (11.2 m from the test well), and LAR-1 (13.4 m from the test well) during an interference slug test conducted with straddle packers in fractured biotite gneiss.

2.6 CONCEPTUAL MODEL FOR RWL FLUCTUATIONS EARLY IN WELL TESTS

Reverse water level fluctuations at monitoring wells characterize the initial response of most slug tests conducted at our site. The magnitude of the RWL is on the order of cm of head and this can be nearly the magnitude of the pressure increase during the slug test. Despite the common occurrence and relative magnitude of the RWL response, theoretical methods typically used to analyze slug tests fail to predict this behavior. This deficiency apparently occurs because conventional analyses are
based on solutions of the diffusion equation. Although this equation is widely used to analyze transient problems involving groundwater flow, it uses a simplified approach to evaluate how pressure changes deform the solid framework of the aquifer. The RWL observed during our tests is inferred to result from the coupling of displacement and pressure change that differs from the assumption used in the diffusion equation.

A conceptual model of this process can be visualized by independently examining the effects of primary and secondary sources of pressure change. These two sources of pressure change occur as a coupled process, but it will initially be convenient to separate them and discuss each independently.

2.6.1 Primary pressure component

A well test involves increasing or decreasing fluid pressure by adding or removing water from the fracture. This perturbation in pressure is initially greatest at the well and it propagates away from the well with time. The change in pressure diminishes with distance and beyond the leading edge the change in pressure will essentially be zero. It is assumed in Figure 2.11 that the walls of the fracture are rigid, so the water compressibility and geometry control the pressure distribution. This will be called the “primary” pressure change caused by the well.
Figure 2.11 Conceptual model of the development of a reverse water level fluctuation in a fracture during a well test. a.) primary pressure change caused by well test extends a finite distance b.) primary pressure change displaces fracture walls beyond the pressure leading edge. This creates a secondary pressure change of opposite sign. c.) observed result is combination of a.) and b.), resulting in a reverse fluctuation (blue) in front of the pressure increase caused by the well.

Consider as an example the pressure distribution resulting from a well test involving a constant rate of injection or recovery, $Q$. The primary pressure ($P$) relative to ambient will be distributed after an initial early-time period as

$$\Delta P_i = \frac{Q \gamma}{4 \pi T} \ln \left( \frac{4 T \tau}{r_w^2 S} \right)$$  \hspace{1cm} (2.1)
where $T$ is effective transmissivity, $S$ is storativity, and $\gamma$ is unit weight of water. We pick a value of pressure $\Delta P_1$ to define the leading edge of the pressure, which occurs at $r = r_L$. It follows that the leading edge occurs at

$$r_L = \sqrt{\frac{4Tt}{S} e^{-\frac{\Delta P_1 4eT}{2Q}}} = 2\sqrt{D_t} e^{-\frac{\Delta P_1 3eT}{6Q}}$$

(2.2)

The rate of advance of the leading edge of the pressure is the derivative of equation 2.2

$$V_p = C_1 \sqrt{\frac{D_h}{t}}$$

(2.3)

where the hydraulic diffusivity $D_h$ for a single rigid fracture is taken as

$$D_h = \frac{\delta^2}{\beta 12 \mu}$$

(2.4)

and

$$C_1 = e^{-\frac{\Delta P_1 3m^3}{Q 6 \mu}}$$

(2.5)

where $\delta$ is fracture aperture, $\mu$ is viscosity of water, $\beta$ is compressibility of water.

Consider a fracture with $\delta = 2.30 \times 10^{-4}$ m, $\mu = 0.001$ Pa s, $\Delta P_1 = 10$ Pa, $Q = 10^{-4}$ m$^3$/s, $\beta = 4.4 \times 10^{-10}$ Pa$^{-1}$. In this scenario, $D_h = 10^4$ m$^2$/s and $C_1$ is essentially unity. As a result, $V_p = 30$ m/s when $t = 10$ seconds and it decreases to 3 m/s after 1000 seconds.

The example outlined above considers a constant-rate pumping test. The velocity of the leading edge of the primary pressure will be unaffected by the onset of recovery when $Q$ goes to zero. Slug tests can be represented as a brief period of injection at constant rate followed by recovery. As a result, equation 2.3 gives the
velocity of the leading edge of the pressure caused by a slug test as well as a pumping test.

### 2.6.2 Secondary pressure component

Pressure change during a well test will load the walls of the fracture and cause displacement. A pressure increase pushes the walls of the fracture outward and, in general, the displacement will always extend beyond the leading edge of the pressure as a result of the faster propagation of the mechanical stress than the pressure wave. This can be visualized by recognizing that the spatially variable pressure distributed over the fracture walls can be represented as an array of discrete patches loaded by the average pressure within each patch. The displacement due to each patch is summed to get the total displacement.

Pursuing this approach further, it is assumed that the wall of the fracture behaves as a half space and this allows the displacement due to a circular patch of radius $R$ loaded by a uniform pressure to be evaluated for regions within the patch ($r < R$) as [Davis and Selvadurai 1996; Fig. 4.6]

$$v(r) = \frac{4(1-v^2)PR}{\pi E} E\left(\frac{2}{\pi}, r/ R \right) \quad (r \leq R)$$  \hspace{1cm} (2.6)

where

$$E\left(\frac{2}{\pi}, r/ R \right) = \int_0^\frac{2}{\pi} \sqrt{1 - \left(\frac{r}{R}\right)^2 \sin^2 \Omega} \, d\Omega$$  \hspace{1cm} (2.7)
is the incomplete elliptic integral of the second kind. The displacement outside the
patch can be approximated by assuming the pressure acting over the circle is applied as
a point load, which gives

\[ \nu(r) \approx C_1 \frac{PR^2}{E} \frac{1}{r} \quad (r >> R) \tag{2.8} \]

where \( C_1 \) is a constant [Polous and Davis 1974, 2.1f and 2.2f].

The well tests conducted for this work will change the pressure head over
regions where \( R \) is roughly 10m or greater. Based on this scale, we consider a region of
\( R = 1 \) m to be small enough to assume the pressure is uniform. Assuming a pressure of \( P \)
\( = 10^4 \) Pa applied and \( E = 10^{10} \) Pa, we get the displacement at the edge of this region (\( r = R \)) as [Davis and Selvadurai 1996; Fig. 4.4]

\[ \nu_{pr} \approx \frac{PR}{E} = 10^{-6} \text{ m} \tag{2.9} \]

The displacement decays as \( 1/r \), however, so at \( r = 2 \) m, \( \nu_{pr} \approx 0.5 \times 10^{-6} \) m and at \( r = 10 \) m,
\( \nu_{pr} \approx 10^{-7} \) m.

It follows from equation 2.8 that displacement of the walls of the fracture will
occur beyond the leading edge of the primary pressure, and it appears from the
example that displacement may occur many meters ahead of the leading edge.
Displacements in elastic materials move at the speed of sound in the material, which is
approximately 6000 m/s in granite. This is several orders of magnitude faster than \( \nu_{pr} \), so
we infer that even though the leading edge of the pressure is advancing, displacement
caused by this pressure will always occur beyond the leading edge of the primary pressure.

Displacement of the fracture walls will change the pressure in the fracture. Outward displacement or dilation of the fracture walls will drop the pressure, whereas inward displacement of the walls will increase it. The magnitude of the pressure change will depend on the rate of normal displacement of the fracture walls, so rapid dilation of the fracture will drop the pressure more than a slow dilation will. Displacement is caused by the primary pressure change, so we will call this component the secondary pressure change. In general, the sign of the secondary pressure change will be opposite that of the primary pressure change.

Early in a slug-in test, the fracture walls will open and dilation (positive $d\delta/dt$) will cause the pressure to decrease (Figure 2.11b). Displacement occurs beyond the leading edge of the primary pressure, so the secondary pressure drops in this region (Figure 2.11b).

### 2.6.3 Reverse Water Level Changes in Piezometers

The primary pressure increase from the well test and the secondary pressure decrease from the displacement can be conceptually superimposed to produce the RWL effect observed in the field. The secondary pressure change caused by the displacement reduces the pressure increase caused by the well and it creates the RWL effect in front of the leading edge.
A well test with a RWL is caused by the combination of the two components outlined above. The combination provides a meaningful conceptualization of the process, but it is not a strict superposition involving addition of the primary and secondary pressure (Figure 2.11). The fluid flow and displacement processes mutually interact, so they must be analyzed simultaneously and cannot be simply superimposed. The technique is to develop coupled analyses for dependent variables of fluid pressure and displacement. This problem was solved using the DFrx code [Murdoch and Germanovich 2006].

2.7 DATA INTERPRETATION – NUMERICAL MODEL (DFRx)

Theoretical analyses of interference slug tests typically assume the tests start when the maximum pressure is applied to the wellbore, and they predict the drop in pressure after the peak [Cooper et al. 1967; Ferris et al. 1980; Butler et al. 1996], whereas analyses of pulse tests are able to predict the rise and fall of the pressure head [Grigham 1970; Daltaban and Wall 1998; Hocking 2001]. Typical analyses of slug tests assume storage changes are proportional to the local change in pressure, but in some analyses pressure and deformation are more tightly coupled [Chapuis 1998]. Based upon the conceptual model, it was suspected that the decrease in pressure associated with the RWL was a result of deformation that occurred ahead of the outward spreading pressure disturbance resulting from the slug test. To test this idea, an analysis was developed that couples fluid flow along a fracture to deformation of the fracture walls.
A numerical model, DFrx [Murdoch and Germanovich 2006] was used for this analysis. This model simulates deformation of a single, horizontal, deformable fracture intersected by a vertical well subjected to a change in fluid pressure (Figure 2.12). The fracture is circular and axial symmetry is assumed.

The model considers a well bore intersected by a finite-length horizontal fracture which is assumed to be fluid-filled and partially open with portions of the opposing fracture walls in contact on asperities. The direction of least principle compression is assumed to be oriented normal to the fracture plane, neglecting shear displacements [Murdoch and Germanovich 2006]. The analysis is conducted by calculating displacements by applying the loads from fluid pressure ($P$), and the effective stress on the asperities ($\sigma_e$) to the walls of a cavity in a compressible matrix [Sneddon 1995]. A uniform compressive stress ($\sigma_c$) is also assumed to oppose fluid pressure and effective stress within the fracture [Murdoch and Germanovich 2006]. A finite difference scheme is used to calculate the pressure distribution, which requires knowing...
the displacement since the last time step. Thus, the fluid pressure and displacements are coupled and the overall problem is solved using a non-linear equation solver. Details are explained in Murdoch and Germanovich [2006]. Effects of cross-cutting fractures, and blockage in the fracture are incorporated into the model to represent field conditions.

Fluid flow through the fracture is assumed to be laminar and is represented using [Witherspoon and Wang 1980]

\[ q = -\frac{\delta^2 \gamma}{12 \mu} C_f \frac{dh}{dr} \]  

(2.10)

where \( \gamma \) is the unit weight of water, \( \mu \) is the dynamic viscosity of water, and \( C_f \) is a friction coefficient that accounts for the tortuosity of fluid flow through a partially open crack [Renshaw 1995; Zimmerman and Main 2004; Cook et al. 1990; Walsh 1981].

Blockage will locally reduce the transmisivity of the fracture and are included by adjusting \( C_f \) at a radial distance from the source well. A blockage term is used to describe the effects of secondary mineral precipitation (e.g. silica, calcite, or clay minerals) or increased density of contacting asperities that would impede flow through the fracture.

The leakage flux out of the fracture is represented using

\[ q_L(r) = 2C_w (h - h_0) \]  

(2.11)

where \( r \) is the radial distance from the center of the wellbore, \( C_w \) is the conductance of the fracture wall, and \( h_0 \) is assumed to be constant [Murdoch and Germanovich 2006].
A leakage term is used to represent the effects of a permeable zone, such as a vertical fracture, that cuts across the idealized flat-lying fracture intersecting the wellbore by increasing the value of $C_w$ at a radial distance from the test well represented by the designated grid block.

The driving pressure ($P_d$) for the fracture is the difference between the sum of the weighted fluid pressure ($P$) and effective stress ($\sigma_e$) that open the fracture, and the confining stress ($\sigma_c$) that closes it according to

$$P_d(r) = \alpha P + \sigma_e - \sigma_c$$

where $\alpha$ is defined as the ratio of the area of open space to the total area of the fracture surface. Using this form of $P_d$ allows for apertures of asperity supported fractures in partial contact to be solved using elastic displacement equations [Murdoch and Germanovich 2006].

The fracture is assumed to deform globally with displacement occurring in one location along the fracture in response to a change in driving pressure at another location on the fracture face [Murdoch and Germanovich 2006]. This distinguishes the analysis used here for a deformable fracture from other models that simulate an aquifer consisting of equivalent porous media where deformation is only local [Schweisinger et al. 2007].

For relating displacement and effective stresses applied to the fracture, the aperture is assumed to be approximated as a linear function of effective stress in the equation
\[
\delta_L(r) = \delta_o - \frac{\sigma_e}{k_n}
\]  

(2.13)

where \(\delta_L(r)\) is the local aperture some radial distance from the wellbore, \(\delta_o\) is the aperture when effective stress is zero, and \(k_n\) is the normal stiffness of the fracture.

Assuming the driving pressure in equation 2.12 is distributed over a circular crack in an infinite medium allows the displacements to be determined by [Sneddon 1946]

\[
\delta_g(r) = \frac{8a(1-\nu^2)}{\pi E} \frac{1}{r/a} \int_{\mu / a}^{1} \frac{\mu}{(\mu^2 - r / a)} d\mu \int_{0}^{\frac{\pi}{2}} \frac{P_d(\tau \mu a)}{\sqrt{1-\tau^2}} d\tau
\]

(2.14)

where \(\nu\) is Poisson’s ratio. The fluid pressure and displacements are coupled during hydraulic testing because increasing the fluid pressure changes the aperture, but changing the aperture affects the pressure distribution during fluid flow. The local effect assumes aperture changes vary linearly with effective stress, as described in equation 2.13. The global effect is characterized by elastic deformation as in equation 2.14. Coupling requires

\[
\delta_L(r) = \delta_g(r)
\]

(2.15)

An in depth description of the theoretical analysis behind DFrx can be found in Murdoch and Germanovich [2006].

2.7.1 DFrx Baseline Analysis

A baseline slug test was simulated by elevating the hydraulic head as a linear function of time for 5 seconds and then allowing it to recover. Parameters used were
typical for conditions at our field site and are presented in Table 2.2. A blockage term was not included in the baseline analysis. The results show that pressure and displacement propagate rapidly away from the well, with effects extending 10 m or more when the maximum pressure is reached at 5 sec into the test (Figure 2.13).
Figure 2.13 Radial profiles of hydraulic head, displacement and normalized displacement rate as functions of time during a slug-in test. The full profiles are shown on the left side, the magnified profiles showing details of the RWL on the right side. Colors correspond to the conceptual model presented in Figure 2.11.
Table 2.2: Baseline DFrx Model Input Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic Modulus, $E$</td>
<td>30.0E+09</td>
<td>Pa</td>
</tr>
<tr>
<td>Poisson’s Ratio, $\nu$</td>
<td>0.3</td>
<td></td>
</tr>
<tr>
<td>Fracture Normal Stiffness, $K_{ni}$</td>
<td>1.0E+09</td>
<td>Pa/m</td>
</tr>
<tr>
<td>Initial Fracture Aperture, $\delta_0$</td>
<td>6.00E-4</td>
<td>m</td>
</tr>
<tr>
<td>Contact Area Ratio, $\alpha$</td>
<td>0.70</td>
<td></td>
</tr>
<tr>
<td>Well Casing Radius, $r_c$</td>
<td>0.02540</td>
<td>m</td>
</tr>
<tr>
<td>Wellbore Radius, $r_{cw}$</td>
<td>0.0762</td>
<td>m</td>
</tr>
<tr>
<td>Initial Head, $h_0$</td>
<td>17.4</td>
<td>m</td>
</tr>
<tr>
<td>Confining Stress, $\sigma_c$</td>
<td>5.00E-5</td>
<td>kPa</td>
</tr>
<tr>
<td>Fracture Radius, $r$</td>
<td>500</td>
<td>m</td>
</tr>
<tr>
<td>Matrix Conductance, $C_w$</td>
<td>1.0E-30</td>
<td>Sec(^{-1})</td>
</tr>
<tr>
<td>Distance to Leakage, $r_L$</td>
<td>464</td>
<td>m</td>
</tr>
<tr>
<td>Matrix Conductance at $r_L$, $C_{wL}$</td>
<td>10E-2</td>
<td>Sec(^{-1})</td>
</tr>
</tbody>
</table>

The hydraulic head drops from a maximum of 4 m at the well to small values at $r = 10$ m, whereas displacement decreases from roughly 3 mm at the well to several tenths of a micron at $r = 10$ m. Examining the head profile in detail shows that when $t = 5$ sec, $h$ drops to 0 at $r \approx 11.5$ m and it reaches a maximum negative value of less than -0.02 m at $r \approx 18$ m. This region of negative $h$ is the RWL. It appears from the simulation that the RWL region extends beyond $r \approx 30$ m at $t = 5$ sec. The RWL appears to span a radial distance longer than that where the heads are elevated at $t = 5$ sec, according to the simulations.

The displacement caused by the slug test decreases from a maximum of approximately 3 microns at the well to 0 at $r \approx 22$ m. The normalized displacement rate $(1/\delta)dw/dt$ reaches a maximum of $10^{-3}$ s\(^{-1}\) and it also decreases to 0 at $r \approx 22$ m.
Most of the RWL occurs at the leading edge of the region where the fracture is dilating in response to the slug test (Figure 2.13). This response is apparent at $t = 5$ sec, but the same general arrangement of head, displacement, and displacement rate occurs throughout the test, with the magnitudes and locations changing with time (Figure 2.13).

The simulations confirm the qualitative conceptual model presented earlier, where the RWL occurs as a response to displacements caused by the pressure change near the well. The simulations show the effect of increasing pressure heads in the well, but a similar effect occurs when the pressure heads are dropped, as in a slug-out test.

It is also apparent that effects not anticipated in the qualitative model occur when the coupling between pressure and displacement is simulated. The most notable effect occurs at the leading edge of the RWL where the fracture is closing, as indicated by negative displacements in Figure 2.13. The displacement rate is also negative, although this requires careful scrutiny. Apparently, the leading edge of the fracture closes slightly in response to the pressure drop in the RWL before it opens in response to the pressure increase created at the well.

The maximum amplitude of the RWL increases with distance from the well, reaches a maximum and then decreases at greater distances. In the baseline simulations, for example, the amplitude reaches a maximum of $-0.024$ m at $r = 15$ m and it decreases to roughly one tenth of this value at $r \approx 100$ m. Interestingly, the amplitude drops as the source well is approached, so the RWL may be too small to detect at
observation points in close proximity to the source of the slug. The arrival time of the RWL increases with radial distance, and analysis of the data in Figure 2.14 indicates that the distance is proportional to the square root of time, a result consistent with equation 2.2.

![Graph showing magnitude and arrival time of the RWL as functions of radial distance](image)

**Figure 2.14** Magnitude and arrival time of the RWL as functions of radial distance for the baseline case shown in Figure 2.13.

**Comparison to conventional analyses of well tests**

Including the effects of deformation will reduce the pressure at a particular location and it will delay the arrival of pressure waves in the same manner that increasing aquifer compressibility or specific storage will slow pressure transients. However, the effects of deformation that cause the RWL and that are shown in Figure
2.11 are fundamentally different from those of specific storage. The difference is that a point load of pressure only changes storage at that point when specific storage is used, whereas a point load changes storage in its vicinity in the conceptualization used here (Figure 2.11). Specific storage is typically defined as the change in volume stored per unit volume per change in pressure head, which in a fracture becomes

\[ S_s = \frac{dV_s}{Vdh} = \frac{1}{\delta} \frac{dv_{ps}}{dh} \]  

(2.16)

where water compressibility is ignored. Using equation 2.8 in 2.16 gives

\[ S_s \approx C_i \frac{\gamma R^3}{\delta E} \frac{1}{r} \]  

(2.17)

This result shows that rather than being a material property, specific storage from the pressure applied at a point on a fracture is a response whose magnitude varies as 1/r from the point where a pressure is applied. A pressure distribution represented as an integral of point loads would have a related response, where the storage change at any location would have contributions from the conditions in the general vicinity.

2.7.2 DFrx Sensitivity analysis

Analyses of slug tests using DFrx shows that it predicts the RWF using baseline parameters typical of the field site. This was encouraging, so a sensitivity analysis was conducted to investigate the influence of several mechanical and hydrogeologic model
parameters on the pressure response in a monitoring well. These parameters include fracture aperture, fracture normal stiffness, elastic modulus, contact area, leakage, and blockage.

The analysis was conducted using the baseline set of model parameters (Table 2.2) consistent with inferred site conditions. The parameter sensitivity analysis was performed by generating results from several simulations where a single parameter was systematically varied within a specific range. The remaining properties were unchanged from the baseline values during the analysis. A monitoring well at a radial distance of \( r = 20 \text{ m} \) was assumed for the analyses.

Data for each simulation were plotted together to evaluate the pressure response caused by the parameter variation. The scale of the graphs used in the comparisons was determined by both the baseline results and the results of variations in parameter values. Two graphs were plotted for each sensitivity parameter, one graph showing the entire pressure response at the monitoring well, and the other detailing the RWF response.

**Fracture Aperture**

The loaded fracture aperture is calculated by relating the effective stress applied to the initial fracture aperture and the displacement in equation 2.13. The sensitivity analysis indicates that decreasing the fracture aperture (\( \delta_L \)) from 500 microns to 300 microns causes the amplitude of the RWF to increase, occur later in time, and spread over a longer period of time (Figure 2.15a). For example, reducing the aperture from
500 microns to 300 microns causes the RWF amplitude to increase from less than 0.01 m to nearly 0.03 m and the trough arrival time to increase from roughly 2 seconds to more than 8 seconds. The time span of the pulse (duration of the half amplitude) is less than 2 seconds for the largest fracture aperture (500 microns), but it spreads to roughly 5 seconds for the 300 micron fracture. Interestingly, reducing the aperture to less than 300 microns creates an opposite effect, resulting in a decrease in RWF amplitude, with a delay in the trough arrival time. It appears that the RWF amplitude reaches a limit between 300 microns and 200 microns in the DFrx analysis using typical field parameters.

**Fracture Normal Stiffness**

Simulations for the fracture normal stiffness ($k_n$) were conducted for values ranging from $7.5 \times 10^8$ Pa/m to $5.0 \times 10^9$ Pa/m with an applied confining stress of $5.0 \times 10^5$ Pa. In-situ and laboratory determinations of fracture normal stiffness evaluated at effective stresses less than 5 MPa generally range between 1 and 60 GPa/m [Bandis et al. 1983; Swan 1983; Martin et al. 1990]. Because of the way that the DFrx simulation is set up, a change in the fracture normal stiffness in equation 2.13 creates a change in the loaded fracture aperture. As a result, two separate fracture normal stiffness sensitivity simulations were conducted. In one, the fracture normal stiffness was varied while the initial contact aperture was fixed at 700 microns, whereas in the other the normal stiffness was varied while the loaded aperture was fixed. The difference is that in the first case the increase in $k_n$ also causes the loaded aperture to increase, whereas in the
second case $k_n$ was varied between $7.5 \times 10^8$ Pa/m to $5.0 \times 10^9$ Pa/m, while the initial fracture aperture was fixed at 700 microns.

The results for the first case (Figure 2.15b) indicate that increasing the fracture normal stiffness causes the RWL magnitude and arrival time to decrease. The RWL effect disappears altogether when the normal stiffness is large enough. Results for the second case (Figure 2.15c) are similar to, but smaller than for the first case. This suggests that reducing aperture and softening the normal stiffness will have similar effects on the RWF (Figure 2.15).
Figure 2.15  Plots of head as a function of time in a monitoring well located 20 m away from a test well for varying values of (a) fracture aperture, (b) fracture normal stiffness with variable loaded fracture aperture, and (c) fracture normal stiffness with the loaded fracture aperture held constant.


**Elastic Modulus**

The analysis considered four values of Young’s modulus ($E$), ranging from 20 GPa to 50 GPa. This range was selected to represent typical values of modulus for crystalline rock which fall into the range of 1 to 80 GPa as presented by Ide [1936], with unweathered, massive granites having the largest moduli, and weathered and fractured granites having lower moduli. The results show that the overall pressure response is nearly insensitive to changes in $E$ over the specified range, but there is a systematic response in the RWF (Figure 2.16). In particular, increasing $E$ increases the amplitude of the RWF trough, but the trough arrival time is unchanged.

![Graph](image)

**Figure 2.16** Plots of head as a function of time in a monitoring well located 20 m away from a test well for varying values of elastic modulus.

**Leakage**

The leakage term is used in the DFrx model to simulate hydraulic connections between a primary flat-lying fracture and steeply dipping, crosscutting fractures. A sensitivity analysis was conducted on leakage located between the test well and a
monitoring well 20 meters away with leakage introduced at 5 meters, 10 meters, and 15 meters from the test well. A conductance value of $10^{-3.0}$ sec$^{-1}$ was used in a single cell at those designated radial distances. Previous works (Schweisinger 2007) have shown that conductance values greater than $10^{-2.0}$ sec$^{-1}$ behave like constant head boundaries, whereas conductance values less than $10^{-5.5}$ sec$^{-1}$ have negligible effect.

Results indicate that the location of the leakage has an important effect on the RWL (Figure 2.17). Leakage located between the test well and the monitoring point has a significant effect on the RWF, whereas leakage beyond the monitoring well has little effect. The general response is that the amplitude of the RWF is suppressed with leakage closer to the test well. Neither the arrival time nor the width of the RWF appears to be affected.

Interestingly, the positive pressure rise that characterizes the typical response to a slug test is nearly completely suppressed by leakage between the monitoring and test wells in this example. This occurs because the conductance value is large enough for the behavior of the leakage to approach that of a constant head boundary. However, even though the positive pressure resulting from the slug-in is nearly completely suppressed, the RWL is largely unaffected. Moreover, the RWL appears to have remains sensitive to the location of the leakage. This sensitivity to location is suppressed when the leakage is beyond the monitoring point, however, but in these cases the positive pressure rise appears to be sensitive to location (Figure 2.17).
Figure 2.17 Plots of head as a function of time in a monitoring well located 20 m away from a test well. Leakage terms were used to simulate the presence of a vertical fracture crosscutting an idealized horizontal fracture (a) between the test well and monitoring well at 5 meters, 10 meters, and 15 meters away from the test well and (b) beyond the monitoring well at 25 meters, 30 meters, and 35 meters away from the test well.

**Blockage**

A blockage term [Murdoch and Germanovich 2006] is used in the model to simulate a zone of reduced transmissivity within the fracture. The analysis was conducted using a blockage term of 0.01 located at the same radial distances as the leakage zones (r = 5 m, 10 m, 15 m, 25 m, 30 m, and 35 m). The monitoring well was at r = 20 m, as in the previous examples. Results indicate that blockage in the fracture
increases the RWF magnitude and delays the arrival time compared to the baseline model with no blockage (Figure 2.18). These effects increase as the radial distance of the blockage increases and approaches the monitoring well. For example, a blockage at \( r = 15 \text{ m} \) increases the amplitude by a factor of four, from -0.01 m to -0.04 m, and delays the peak RWF arrival from 11 seconds to 16 seconds. The RWF appears to be nearly insensitive to blockage beyond the monitoring well.

The general result is that heterogeneities in the fracture (represented here as leakage and blockage) located between the test well and the monitoring well influence the RWF, whereas the effect on the positive pressure response after the RWF is variable. In contrast, the effects of heterogeneities located beyond the monitoring well appear to be minor, but they do influence the positive pressure response after the RWF.
Figure 2.18 Plots of head as a function of time in a monitoring well located 20 m away from a test well. Blockage terms were used to simulate lower permeability zones within an idealized flat lying fracture (a) between the test well and monitoring well at distances of 5 meters, 10 meters, and 15 meters away from the test well and (b) beyond the monitoring well at 25 meters, 30 meters, or 35 meters away from the test well.

2.8 ANALYSIS OF FIELD DATA

The DFrx model appeared to predict the type of behavior observed in the field, so it was used to analyze a representative field data set (Figure 2.10) where the RWL signal was discernible. This was accomplished using the parameter estimation software PEST, a model-independent program that utilizes the Gauss-Marquardt-Levenberg method to minimize an objective function using nonlinear regression [Waterloo Hydrogeologic 2002]. A similar combination of DFrx and PEST has been used to analyze
hydromechanical slug test data from this site [Svenson 2006; Svenson et al. 2007, Svenson et al. 2008]. However, previous work was limited to datasets obtained from the test well, so they did not include the RWF.

For each simulation, the fracture was assumed to be circular in plan view with a radius of 500 m to ensure that boundary conditions did not affect model results [Schwiesinger 2007]. A confining stress of $5.0 \times 10^5$ Pa was imposed on the fracture and was derived from the product of the average unit weight of the biotite gneiss and saprolite overlying the test interval. A well radius ($r_w$) of 0.076 m and a pipe string radius ($r_c$) of 0.038 were used for the simulation. The increasing head in the test well during initial pressurization of the slug test was simulated by increasing the initial head by 0.524 m per second over an 8 second period. Sentinel head observations along the pressure curve in LAR-4 were assigned a weight of 0.4, while the head observations in the monitoring wells were weighted with an emphasis being placed on the RWF (1.0 weighting) and a weight of 0.8 assigned to the positive pressure response.

For initial simulations, the fracture normal stiffness, initial fracture aperture, elastic modulus, and contact ratio along with parameters controlling leakage and blockage were varied using the PEST program. The PEST optimization procedure requires user defined initial values for each parameter along with upper and lower limits to constrain the parameters during the estimation procedures. Typical initial parameters along with upper and lower limits are indicated in Table 2.3.
Several parameter estimation simulations were conducted, and the model parameter predictions of each simulation were used to narrow the upper and lower bounds, and modify initial parameter values until the model fit to the LAR-4 and closest monitoring well LAR-3 data could not be improved by further estimation runs. The Dfrx model input parameters for the best fit to the field data obtained through the PEST simulations are included in Table 2.4.

Table 2.4: Best fit parameter estimates

<table>
<thead>
<tr>
<th>Parameter</th>
<th>PEST Estimated Value</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic Modulus, $E$</td>
<td>2.80E+10</td>
<td>Pa</td>
</tr>
<tr>
<td>Fracture Normal Stiffness, $K_n$</td>
<td>2.31E+08</td>
<td>Pa/m</td>
</tr>
<tr>
<td>Initial Fracture Aperture, $\delta_0$</td>
<td>8.00E-04</td>
<td>M</td>
</tr>
<tr>
<td>Contact Area Ratio, $\alpha$</td>
<td>0.24</td>
<td>Unitless</td>
</tr>
<tr>
<td>Distance to Leakage, $r_L$</td>
<td>1.20</td>
<td>M</td>
</tr>
<tr>
<td>Leakage Strength, $C_{wL}$</td>
<td>-4.31</td>
<td>Sec$^{-1}$</td>
</tr>
<tr>
<td>Distance to Blockage, $r_B$</td>
<td>10.00</td>
<td>M</td>
</tr>
<tr>
<td>Blockage Strength, $C_{fB}$</td>
<td>4.16E-03</td>
<td></td>
</tr>
</tbody>
</table>

The Dfrx model predicts the general response observed in the field data, where the pressure decreases and then increases before equilibrating (Figure 2.19). The quality of fit between the model predictions and the field data was evaluated by...
calculating the coefficient of determination, \( R^2 \). Values of \( R^2 \) were determined by dividing the sum of residual errors, SSE, by the sum of squares about the mean, SSM, according to the equation

\[
R^2 = 1 - \left( \frac{SSE}{SSM} \right)
\]  

(2.18)

where

\[
SSE = \sum_{i=1}^{n} (O_i - O_p)^2
\]  

(2.19)

and

\[
SSM = \sum_{i=1}^{n} (O_{i\text{mean}} - O_i)^2
\]  

(2.20)

and \( n \) is the number of observations, \( O_i \) is the measured field value, \( O_p \) is the value predicted by the model, and \( O_{i\text{mean}} \) is the mean value of the field measurements.

The error between predicted and observed is relatively low in data from the test well \((R^2 = 0.992)\) and the closest monitoring well \((R^2 = 0.847)\). The RWF is evident in the predicted response for LAR-3, although the predicted RWF magnitude \((-0.01305 \text{ m})\) is larger than that observed \((-0.0085 \text{ m})\). DFrx also has the ability to predict heads at distances into the formation, and the model predicted head was compared to the LAR-2 and LAR-1 monitoring well field responses. DFrx was able to predict the general shape of the RWL fluctuation and positive pressure response at the location of the monitoring wells, however, the maximum amplitude of the RWF and peak positive pressure signal in the field data occurs much later than that predicted by DFrx. Additionally, the
magnitude of the DFrx predicted peak positive pressure response is approximately five times greater than the field data peak positive pressure.

The Pest optimization procedure creates a variance-covariance matrix which is used to derive statistics describing the precision (95% confidence interval) and correlation (parameter correlation coefficients) of the estimated parameters. The 95% confidence interval is a measure of the precision of each parameter value. Narrow confidence intervals indicate greater precision while wider intervals indicate less precision [Hill 1998]. The parameter correlation coefficient ranges between -1.0 and 1.0 and represent the uniqueness of the estimated parameters [Hill 1998]. Correlation coefficients values that are close to -1.0 and 1.0 are indicative of correlated parameters that cannot be uniquely estimated from the field data [Hill 1998; Svenson 2006].
Figure 2.19  DFrx model fit of a typical interference slug test response from the test well (LAR-4), and three monitoring wells located 5.3 meters (LAR-3), 11.2 meters (LAR-2), and 13.4 meters (LAR-4). Field Data are open circles, DFrx model output is indicated with a solid line.

For initial simulations where eight parameters (fracture normal stiffness ($K_n$), initial fracture aperture ($\delta_0$), elastic modulus ($E$), contact ratio ($\alpha$), distance to leakage ($r_l$), leakage strength ($C_{wl}$), distance to blockage ($r_b$), and blockage strength ($C_{fb}$)) were varied, inspection of the variance-covariance and correlation matrices revealed that the parameters yield large 95% confidence intervals and correlation coefficients that are close to 1.0 and -1.0, indicating that that more than one parameter have a similar effect
on the model solution [Waterloo Hydrogeologic 2002]. The estimated parameter values along with their 95% confidence intervals and correlation coefficients are shown in Tables 2.5 and 2.6, respectively.

**Table 2.5: 95% confidence interval; Eight parameters estimated**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Lower Bound</th>
<th>Upper Bound</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic Modulus, $E$</td>
<td>2.80E+10</td>
<td>-1.83E+13</td>
<td>1.84E+13</td>
</tr>
<tr>
<td>Fracture Normal Stiffness, $K_n$</td>
<td>2.31E+08</td>
<td>-3.91E+11</td>
<td>3.91E+11</td>
</tr>
<tr>
<td>Initial Fracture Aperture, $\delta_0$</td>
<td>8.00E-04</td>
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<td>0.51</td>
</tr>
<tr>
<td>Contact Area Ratio, $\alpha$</td>
<td>0.24</td>
<td>-183.01</td>
<td>183.49</td>
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<td>Distance to Leakage, $r_L$</td>
<td>1.20</td>
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<td>92.82</td>
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<tr>
<td>Leakage Strength, $C_{WL}$</td>
<td>-4.31</td>
<td>-99.53</td>
<td>90.91</td>
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<tr>
<td>Distance to Blockage, $r_B$</td>
<td>10.00</td>
<td>-865.04</td>
<td>885.04</td>
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<tr>
<td>Blockage Strength, $C_{FB}$</td>
<td>4.16E-03</td>
<td>-0.76</td>
<td>0.77</td>
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</tbody>
</table>

Confidence intervals are large, and unrealistic in some cases, and correlation is high. Based upon the sensitivity analysis conducted on individual parameters, it was determined that fracture stiffness, fracture aperture, and leakage were the primary parameters that influenced the RWL response during the slug tests. Additional PEST simulations were carried out by fixing all other parameters, and varying only these select parameters ($K_n$, $\delta_0$, $r_L$, and $C_{WL}$) in an attempt to reduce the correlation.
coefficients and 95% confidence interval. Similar approaches have been taken by Schwiesinger [2007] and Svenson [2006]. Two model runs, one that varied $K_n$, $r_L$, and $C_{wl}$, and the other that varied $\delta_0$, $r_L$, and $C_{wl}$ were conducted to try to improve fit and reduce the confidence intervals.

For the PEST simulation where $\delta_0$, $r_L$, and $C_{wl}$ were varied, the parameters $K_n$, $E$, $\alpha$, $r_B$, and $C_{fb}$ were fixed to the values obtained during the earlier best fit parameter estimation runs (Table 2.4). The error between the model prediction and the observed data improved for both the test well ($R^2 = 0.995$) and the closest monitoring well ($R^2 = 0.904$). However, the DFrx model was unable to significantly improve the match to the RWF signal in LAR-3 with a predicted magnitude of -0.0133 m compared to predicted magnitude of -0.0131 m during the Pest model run where all parameters were varied (observed -0.0085 m).

95% confidence intervals for the three estimated parameters ($\delta_0$, $r_L$, and $C_{wl}$) were greatly improved, with a 93 micron range for the $\delta_0$, 0.89 m range for the $r_L$, and 0.17 sec$^{-1}$ range for the $C_{wl}$ (Table 2.7). Parameter correlation coefficients show that $\delta_0$ and $r_L$ are well correlated with a correlation coefficient of 0.815, while the $C_{wl}$ is neither correlated with $\delta_0$ or $r_L$ (Table 2.8).

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Lower Bound</th>
<th>Upper Bound</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\delta_0$</td>
<td>7.42E-04</td>
<td>6.95E-04</td>
<td>7.88E-04</td>
</tr>
<tr>
<td>$r_L$</td>
<td>1.23</td>
<td>0.78</td>
<td>1.67</td>
</tr>
<tr>
<td>$C_{wl}$</td>
<td>-4.29</td>
<td>-4.38</td>
<td>-4.21</td>
</tr>
</tbody>
</table>
Table 2.8: Parameter correlation coefficient; $\delta_0$, $r_L$, and $C_{wL}$

<table>
<thead>
<tr>
<th></th>
<th>$\delta_0$</th>
<th>$r_L$</th>
<th>$C_{wL}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\delta_0$</td>
<td>1.000</td>
<td>0.815</td>
<td>0.160</td>
</tr>
<tr>
<td>$r_L$</td>
<td>0.815</td>
<td>1.000</td>
<td>0.090</td>
</tr>
<tr>
<td>$C_{wL}$</td>
<td>0.160</td>
<td>0.090</td>
<td>1.000</td>
</tr>
</tbody>
</table>

For the PEST simulation where $K_n$, $r_L$, and $C_{wL}$ were varied, the parameters, $\delta_0$, $E$, $\alpha$, $r_B$, and $C_{fB}$ were fixed to the values obtained during the earlier best fit parameter estimation runs (Table 2.4). The error between predicted and observed data for the test well ($R^2 = 0.995$) was slightly improved over the model run where all parameters were varied. The error between the predicted and observed data for the closest monitoring well ($R^2 = 0.903$) was also improved over the model run where all parameters were varied. The DFrx model estimated the RWF signal in LAR-3 with a predicted magnitude of -0.0132 m (observed -0.0085 m).

95% confidence intervals for the three estimated parameters were greatly improved from the case where all parameters were estimated, with a range of 8.10E7 Pa/m for $K_n$, a range of 0.70 m for the $r_L$, and a range of 0.16 sec$^{-1}$ for the $C_{wL}$. Parameter correlation coefficients show that $K_n$ and $r_L$ are fairly well correlated with a correlation coefficient of 0.830, while the $C_{wL}$ is neither correlated with $K_n$ or $r_L$.

Table 2.9: 95% Confidence interval; $K_n$, $r_L$, and $C_{wL}$ estimated

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Lower Bound</th>
<th>Upper Bound</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_n$</td>
<td>2.12E+08</td>
<td>1.75E+08</td>
<td>2.56E+08</td>
</tr>
<tr>
<td>$r_L$</td>
<td>1.05</td>
<td>0.70</td>
<td>1.40</td>
</tr>
<tr>
<td>$C_{wL}$</td>
<td>-4.29</td>
<td>-4.37</td>
<td>-4.21</td>
</tr>
</tbody>
</table>
Based on the two Pest simulations previously described, it appeared that both simulations produced similar results in model fit, 95% confidence intervals, and correlation. A final PEST simulation was conducted by varying only $\delta_0$ and $K_n$, while keeping the parameters $E$, $\alpha$, $r_B$, $C_{fb}$, $r_L$, and $C_{wl}$ fixed to the values obtained during the earlier best fit parameter estimation runs (Table 2.4). The $R^2$ error value between predicted and observed data was 0.994 for the test well and 0.915 for the closest monitoring well.

95% confidence intervals for the estimated parameters were 4.22e-04 m for the $\delta_0$ and 2.08e9 Pa/m for the $K_n$. Parameter correlation coefficients show that the $\delta_0$ and $K_n$ are well correlated with a correlation coefficient of -0.997.

### Table 2.10: Parameter correlation coefficient; $K_n$, $r_L$, and $C_{wl}$

<table>
<thead>
<tr>
<th></th>
<th>$K_n$</th>
<th>$r_L$</th>
<th>$C_{wl}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_n$</td>
<td>1.000</td>
<td>0.830</td>
<td>0.097</td>
</tr>
<tr>
<td>$r_L$</td>
<td>0.830</td>
<td>1.000</td>
<td>0.028</td>
</tr>
<tr>
<td>$C_{wl}$</td>
<td>0.097</td>
<td>0.028</td>
<td>1.000</td>
</tr>
</tbody>
</table>

### Table 2.11: 95% Confidence interval; $K_n$ and $\delta_0$ estimated

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Lower Bound</th>
<th>Upper Bound</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_n$</td>
<td>6.97E+08</td>
<td>2.13E+08</td>
<td>2.29E+09</td>
</tr>
<tr>
<td>$\delta_0$</td>
<td>5.04E-04</td>
<td>2.93E-04</td>
<td>7.15E-04</td>
</tr>
</tbody>
</table>

### Table 2.12: Parameter correlation coefficient; $K_n$ and $\delta_0$

<table>
<thead>
<tr>
<th></th>
<th>$K_n$</th>
<th>$\delta_0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_n$</td>
<td>1.000</td>
<td>-0.997</td>
</tr>
<tr>
<td>$\delta_0$</td>
<td>-0.997</td>
<td>1.000</td>
</tr>
</tbody>
</table>
2.9 DISCUSSION

Reverse water level fluctuations during slug tests appear to be an unrecognized phenomena that occur in fractured rock, and perhaps in other formations. The coupling between the deformation of fracture walls and the changes in pressure apparently causes the RWF. However, the coupling of these conditions is omitted from commercially available models. The analysis using DFRx enables the coupling of both deformation and fluid pressure change within the fracture, and provides insight into causes of the RWF in fractured rock.

2.9.1 DFRx Model Parameter Uncertainty

Confidence intervals about the estimated parameters are large because key parameters are correlated during the forward simulation. This is a consequence of the way the analysis implemented by the DFRx code is formulated. The analysis assumes that the aperture of a fracture is caused by a driving stress, which consists of contributions from water pressure and from effective stresses on surface asperities that are in contact with the opposing fracture surface. As a result, the aperture cannot be specified at the beginning of an analysis. Instead, it must be calculated during an initialization step based on the initial fluid pressure, confining stress, and fracture normal stiffness parameters. Two key parameters, the fracture normal stiffness, $k_n$, and the initial contact aperture, $\delta_o$, are strongly correlated during the initialization when the initial fracture aperture must satisfy
\[ \delta = \delta_o - \sigma \gamma / k_n \] 

(2.21)

Increasing \( \delta_o \) will have the same effect on \( \delta \) as increasing \( k_n \), according to equation 2.21. This response is much different than the effect that \( \delta_o \) and \( k_n \) have on the behavior of the simulation after initialization has taken place. This can be seen in the sensitivity analysis (Figure 2.15c) where varying \( k_n \) while holding \( \delta \) constant causes a much subdued change in the RWF response compared to allowing \( \delta \) to vary. These effects are significantly different and this allows the parameter estimation process to identify values of \( \delta_o \) and \( k_n \) that reduce the objective function towards a minimal value. To further improve confidence, each parameter estimation analysis was conducted with many different initial parameter estimates in order to reduce the potential effects of local minima in the objective function.

The result is that the 95% confidence intervals given by PEST appear to be poor estimates of the parameter uncertainty for this problem. We evaluated the feasibility of modifying the DFrx code to eliminate the initialization step, but this would require major modifications and appears unwarranted. A better alternative seems to be to use a different forward model altogether. For example, finite element methods can be used that allow the initial aperture of a fracture to be assumed a priori [Murdoch et al. 2009]. This eliminates the need for initialization and reduces the confidence intervals on estimated parameters. Alternative techniques of parameter estimation that are based on Monte Carlo methods [Metropolis and Ulam 1949] may provide better evaluations of parameter uncertainty using the DFrx code.
2.9.2 Field Data Differences

Despite inherent model parameter uncertainties introduced by the DFrx model analysis setup, differences between the model prediction and the field data in the monitoring wells cannot be fully explained by the model setup alone. The DFrx model has the ability to match the LAR-3 signal with relatively accurate results ($R^2 = 0.915$). However, the error is much greater for LAR-2 and LAR-1 where the maximum amplitude of the RWF and peak positive pressure signal in the field data occurs after that predicted by DFrx. Additionally, the magnitude of the DFrx predicted peak positive pressure response is approximately five times greater than the peak positive pressure of the field data.

One possible explanation of the differences between the modeled values and field data is that the DFrx model assumes a single flat lying fracture, but in the actual field setting, the complexity of fracture networks and interconnectivity among naturally occurring fractures can create pathways that behave differently than the one that was assumed. These indirect flow paths could delay the arrival times and decrease the magnitude of the slug test pressure pulse. Pressure responses at shorter and longer distances from the test well were evaluated using the DFrx model in an attempt to represent an indirect pathway as opposed to the single flat lying fracture, however, additional simulations were unable to account for the large delays between the model output and the field data and the differences between magnitudes.
Another possible explanation in the differences between the modeled and field data is that the storage of the LAR-3 borehole between the test well and LAR-2 and LAR-1 may be acting like a sink. The effects of storage from open boreholes has been incorporated into traditional interference slug test analyses [Butler and Zhan 2004], but is often assumed to be negligible during straddle packer testing [Delleur 2007]. However, the fracture network may allow leakage around the packer, which would allow the water level in the wellbore to change and this could significantly increase the volume of fluid that could be released from storage.

To further investigate the influence of wellbore storage on the interference pressure response, the DFrx parameters from the best fit to the field data (Table 2.4) were manipulated to include a second leakage source at 5.3 meters from the test well. The additional leakage source was assigned a strength of $-4.0 \text{ sec}^{-1}$, and the DFrx model was reran. The resultant pressure response in LAR-1 and LAR-2 are shown on Figure 2.20.
Figure 2.20  Plots of field results and DFRx analysis results. DFRx analysis results from the best PEST simulation (Table 3) are represented with solid black lines. Dfrx results when a leakage term was added at the location of LAR-3 to simulate well-bore storage are shown as dashed lines. Field Data is represented by open circles.

The plots show that the inclusion of an additional leakage term significantly reduces the magnitude of the positive pressure response signal in LAR-2 and LAR-3 to closer to that measured in the field. An increase in leakage at the approximate location of the LAR-3 borehole reduces the magnitude of the pressure response to roughly the correct value, but has no effect on the arrival time of either the RWL or peak positive pressure response. An additional effect is required to explain the delayed arrival time.

One possible effect that would explain the delayed arrival time is the presence of a preferential pathway between LAR-4 and the closest monitoring well, LAR-3, perhaps due to a larger fracture aperture between the LAR-4 and LAR-3 boreholes. A smaller fracture aperture beyond the LAR-3 borehole could cause the delay in the pressure signal similar to that observed during the DFrx sensitivity analysis where fracture aperture was reduced below 300 microns. The DFrx code is currently being modified to
include an additional parameter that will reduce the fracture aperture beyond the
distance of the LAR-3 borehole.

2.10 CONCLUSION

Measurements of pressure in monitoring wells completed in fractured gneiss show a characteristics rise and fall that is typical of interference slug tests in other aquifer materials. This signal can be analyzed using available methods to provide insights into formation characteristics derived by assuming the fractured medium is represented as an equivalent porous material. Scrutiny of the pressure records shows that the typical response is commonly preceded by a reverse water level response, where the pressure decreases. This effect, which we call a Reverse Water-level Fluctuation (RWF), has a maximum amplitude of slightly less than 1 to several cm of head, and lasts for a few to several tens of seconds in our field tests. For comparison, the positive pressure response caused by the slug test was as great as 10 to 25 cm in some monitoring wells, and this significant difference between the RWF and the positive pressure response may be why this effect has not been reported by previous investigations.

Even though the RWF effect during slug tests had not been described, there is considerable evidence that it occurs. Four consecutive slug tests produced RWF responses that were essentially indistinguishable from each other (they were within the
noise level of the pressure transducer measurements). This repeatability result was obtained at three different intervals, using two different source pressures.

Not only is the RWF repeatable, it can be explained by a conceptual model where fluid pressure and solid deformation are coupled. The pressure increase during a slug-in test causes fractures to dilate in the vicinity of the well. The region where dilation occurs extends beyond the pressurized region, just as the region opened by a wedge occurs beyond the wedge itself. Fracture dilation causes pressures to drop, so the RWF appears to result from pressure-induced displacement that occurs beyond the region where pressure is directly affected by the slug.

The conceptual model is confirmed by a theoretical analysis of coupled fluid flow and deformation, which predicts the essence of the field observations. The analysis shows that a RWF marks the leading edge of the pressure signal of slug tests in many situations. The magnitude of the RWF depends on properties of the fracture, as well as the distance of the monitoring well from the test well.

Coupling of fluid pressure and displacement are included in the theoretical analysis. The explanation of this effect is that it results from deformation that occurs ahead of the area where the stress on the well has changed the fluid pressure. A positive increase in pressure at the well opens the fracture and this dilation causes the pressure to drop in regions relatively distant from the well.

The conceptual model is represented in a coupled theoretical model, which predicts responses that are remarkably similar to field observations. Sensitivity analyses
indicate that the magnitude of the RWF increases with Young’s Modulus of the rock and the compliance of the fracture (inverse normal stiffness). Decreasing the aperture of the fracture delays the arrival, but it has a variable effect on the amplitude. Decreasing aperture from 500 microns to 300 microns causes the amplitude of the RWF to increase, whereas shrinking aperture to less than 300 microns reduces the amplitude of the RWF. Interestingly, the location of the monitoring well also has a variable effect on amplitude: the amplitude increases as the well is approached from a far distance, but the amplitude reaches a maximum value when the monitoring well is some distance (15 m in the example) from the source well. The amplitude of the RWF decreases at monitoring wells closer than this distance. This leads to the surprising conclusion that the RWF may be too small to detect when the monitoring well is either too far from, or too close to the source well.

The use of a high resolution pressure transducer in an interference slug test set-up reveals a reverse water level fluctuation signal previously not documented. Previous works have used conventional slug test analysis techniques to analyze interference slug tests and provide more accurate information of the hydrogeological characteristics of the formation between wells. However, the presence of a reverse water level fluctuation in interference slug tests conducted in a fractured biotite gneiss indicates a poroelastic response not accounted for in conventional analysis techniques. The DFrx analysis used for this work is able to account for the coupling of hydrogeological and mechanical interaction between the rock and groundwater and
readily predict the reverse water level fluctuation. However, disparities between the DFr model prediction and actual field data are apparent. The difference between the DFr model prediction and the field data, particularly in LAR-2 and LAR-1, is most likely attributed to the simplification of a complex fracture network between the test well and monitoring wells into a single, flat lying idealized fracture and wellbore storage in LAR-3. In particular, it is hypothesized that the fracture aperture decreases between LAR-3 and LAR-1, and it is expected that DFr will be capable of matching the RWF response in LAR-1 and LAR-2 when the effect is included.
CHAPTER THREE
INTERFERENCE SLUG TESTS IN A FRACTURED BIOTITE GNEISS: FIELD APPLICATION AND INTERPRETATION

3.1 Overview

Characterization of fracture networks and the identification of hydraulically active fractures that dominate flow between wellbores is a basic aspect of understanding the hydrology of fractured rock. The current approach to this type of characterization is to conduct straddle packer tests in order to isolate and identify fractures that create hydraulic connections through a network. However, exploring all of the possible connections between wellbores can require a huge number of straddle packer tests, amounting to significant time and expense. As a result, thorough characterization of fracture networks is often omitted from environmental characterizations, and this can hinder understanding of the flow system and reduce effectiveness of remedial treatments.

The objective of this research is to develop and evaluate a protocol for identifying hydraulically active connections between wellbores that is intended to reduce the time and expense compared to conventional methods. The protocol begins with an initial interference slug test conducted with a low-pressure packer inside the casing of each monitoring well to obtain a total effective transmissivity, $T$, between wellbores. From this point, a Modified Line Search (MLS) method is used to relocate the monitoring well packers. The MLS method involves dividing the length of the well-bore or the number of fractures beneath the packer by approximately half, relocating the
packer and repeating the test. The resulting \( T \) from the lower half of the borehole is compared to \( T \) from the entire bore, and then \( T \) from the upper half of the bore is estimated by subtraction. The packer is then moved to the middle of either the upper or lower half of the borehole, whichever has the higher \( T \). The process is repeated to determine the most conductive section of the borehole. The MLS approach appears to work well for locating highly conductive zones, but tests with the packer at other locations (further line search divisions) are required where multiple conductive zones are present. This approach is used simultaneously in multiple monitoring wells to highlight the conductive architecture between the source well and the monitoring wells. The sequence is then repeated using a different well as the source.

The MLS methodology can be used to identify sections of the wellbore that are hydraulically dominant, and the number of divisions during the line search method can be increased or decreased depending upon time and budget constraints. Straddle packers are then used to further divide the conductive intervals, isolating individual fractures or fracture sets.

This methodology was applied to wells drilled in a fractured biotite gneiss aquifer at the Clemson University well field. The KGS slug test solution [Hyder et al. 1994] was used to estimate \( T \) between the test well and sections of each monitoring well bore. Two roughly flat-lying transmissive zones were identified with the MLS method at depths ranging from approximately 32 m to 40 m and 40 m to 60 m. Other connections were identified at shallower depths between pairs of wellbore, however, the
connections did not appear to be continuous between all monitoring wells based upon the data obtained using the MLS method. Straddle packer tests were then used to refine the resolution and identify individual or pairs of fractures that were connected to form the transmissive zones.

### 3.2 INTRODUCTION

Flow in fractured rock is often complex, and can create uncertainties when interpreting the results of hydraulic testing of aquifers. Individual fractures can act as flow paths or channels that dominate flow, and can be interconnected to other fractures creating complex, heterogeneous formations \[Day-Lewis et al. 2000\]. Single-well tests, such as the conventional slug test \[Butler 1997\], provide only a broad idea of the average transmissivity adjacent to the well. The conventional analysis of the pressure response in the source well \[Butler 1997\] lacks the resolution to identify fractures controlling or dominating flow, and gives properties that are biased to close proximity of the well \[Belitz and Dripps 1999\]. One consequence of this is that single-well slug tests can give \( T \) values that are dominated by well skin \[Chirlin 1990\]. The use of a pressure signal from one or more monitoring wells (interference slug test) can remedy the shortcomings of the single-well test \[Belitz and Dripps 1999; McElwee et al. 1995a; McElwee et al. 1995b; Spane 1996\].

The interference slug test method requires two or more wells and the test is conducted by rapidly raising or lowering the head in the source well and then measuring
the response at one or more observation wells [Black and Kipp 1977; Spane 1996; Butler 1997]. This measured response in the monitoring well can be analyzed to provide information beyond that of the single well test, and can be critical in reducing uncertainty in the flow and transport of fractured rock systems [Day-Lewis et al. 2000]. Interference slug tests using packers provide a simple method for estimating the hydraulic properties between two straddled intervals. A test can be conducted with packers in one location, and then the packers can be moved and the test repeated to provide another set of data. The distance between the packers is typically small enough to isolate a small number of fractures, so repeating this test many times will eventually identify the connections between individual fractures.

The problem with this approach is that it may require many tests before connections between permeable fractures can be identified. For example, fractures occur along approximately 30 m of 4 boreholes at the Clemson well field. There are 15 possible intervals along each well if the straddle packers are spaced 2 m apart. Characterizing the connections between intervals along two of these boreholes would require \(15^2=225\) tests, and evaluating all the possible combinations using all four wells would require many thousands of tests. It could be possible to eliminate some combinations because they do not intersect permeable fractures, or packers could be used in multiple wells during each test to reduce the total number of tests.

Nevertheless, a thorough evaluation will require many tests and considerable effort. An example of this is described by Karasaski et al. [2000], who used a variety of
characterization techniques including cross-hole radar, seismic tomography, borehole flow surveys, a digital borehole scanner, over 130 injection tests and 4100 cross-hole transient pressure tests incorporating packers, tilt meters, and tracer tests in nine boreholes to identify the permeable network of fractures at the Redmond site. Another example is the Former Loren Air Force Base in northern Maine, USA where Stephenson et al. [2006] used over 100 pulse interference tests to identify fracture connections between well-bores during a small scale steam injection and water vapor extraction pilot study.

The objective of this investigation is to evaluate the feasibility of simplifying the task of identifying permeable connections in fracture networks using packer tests. This will be accomplished by replacing the need to test all the possible combinations of connections with a method for searching large intervals of boreholes for permeable connections, and then progressively refining the resolution with subsequent tests. A technique termed a Modified Line Search (MLS) will be described for this initial characterization. The results of the MLS tests will then be refined using more conventional straddle packer tests and data from other sources, such as video images that help to locate fractures. An application of this approach to a site in western South Carolina will be described.
3.3 GEOLOGIC SETTING

A well cluster located at the Clemson University well field used for aquifer characterization research consists of LAR-2, LAR-3, and LAR-4, which were drilled to 60 m depths and LAR-1, which was drilled to a depth of 120 m (Figure 2.2). Each was cased with 0.15 m diameter casing through approximately 21 m of regolith and is an open borehole to the termination depth of the boring. The dominant rock type is a medium-grained biotite gneiss.

A borehole camera survey was used to map visible fractures within the four boreholes. Numerous fractures and foliation partings were observed during the borehole camera survey and Figure 3.1 presents photographs typical of the fractures observed. Some fractures (Figure 3.1a) were stained reddish brown, apparently from weathering, whereas others (Figure 3.1b) appeared pristine.

![Typical fractures observed in the wellbore camera survey. The orientation and degree of weathering varied between individual fractures and between wellbores. Some fractures (a) are marked by yellow and brown discoloration, apparently from weathering, whereas the rock enveloping other fractures (b) appears pristine.](image)

**Figure 3.1** Typical fractures observed in the wellbore camera survey. The orientation and degree of weathering varied between individual fractures and between wellbores. Some fractures (a) are marked by yellow and brown discoloration, apparently from weathering, whereas the rock enveloping other fractures (b) appears pristine.
The fracture density is 1 to 2 m\(^{-1}\) at a depth of roughly 20 m in LAR-3 and LAR-4, but decreases with depth and is 0.1 to 0.4 m\(^{-1}\) at depths below 40 m. Fracture density is fairly uniform in LAR-2 down to 50 m, however no fractures were observed below that depth. In contrast, fracture density is 0.1 to 0.3 m\(^{-1}\) throughout the entire borehole length of LAR-1 when an averaging length of roughly 5 m is used (Figure 2.3).

3.4 MODIFIED LINE SEARCH PACKER TESTING

Permeable connections between boreholes will be identified by conducting slug tests using the MLS method. The line search method was developed as a global optimization technique originally used as a systematic statistical analysis of neutron multiplication in fission devices [Metropolis 1987]. In our application, the line search method is used to create a systematic methodology for characterizing high transmissivity zones within a wellbore with a limited number of tests [Slack et al. 2006].

The MLS method consists of a series of interference slug tests conducted using a single inflatable packer in each well bore. Low-pressure packers were used for this application because they are considerably easier to deploy than standard, higher pressure packers, and they have been shown to give the same results as higher pressure packers at permeable zones at the Clemson site [Svenson et al. 2005]. The first test is conducted with low-pressure packers located inside the casing of each monitoring well, and in the casing at the test well. The data are analyzed to obtain a total effective $T$ between well bores. The length of each monitoring well bore is then divided in half and
the packer is repositioned at this location. In cases where the fractures are unevenly distributed along the length of borehole, the packer is positioned at the fracture representing the median. The test is repeated and the resulting $T$ from the lower half of each monitoring well is compared to $T$ from the entire bore. $T$ from the upper half of each monitoring well bore is estimated by subtraction. The single packer is then moved to the middle of either the upper or lower half of each monitoring well borehole, whichever has the higher $T$. The process is repeated to identify the most conductive interval connecting the monitoring well and the source well, and the sequence is then repeated using a different well as the source.

### 3.4.1 Field Application

The MLS method field application setup included four vertical monitoring wells (Figure 3.1), one well was used as a source well and the other three wells used as monitoring wells. A single inflatable packer was used in each monitoring well and was placed at the desired depth in each wellbore with a 2-inch-diameter PVC pipe string extending up to the surface. A down-hole pressure transducer was placed below the water level inside the 2-inch pvc pipe string, and sealed using a two-inch diameter inflatable packer located inside the pipe string. Pressurization of the source well casing was controlled using a well-head assembly consisting of a regulator and a series of valves and pressure gauges.
Testing Procedure

The MLS method was used to identify the conductive zones between wellbores within the lower well field. Three separate monitoring wells (LAR-4, LAR-2, and LAR-1) were used as sites of the source well, while the response was measured at each of the remaining three monitoring wells. LAR-3 was not utilized as a test well due to its close proximity to LAR-4.

The line search method initially began with the use of LAR-4 as the test well, and a single packer was placed inside the casing of the test well and inside each of the monitoring wells (Figure 3.2). A slug-in test was initiated in the test well, LAR-4, and the interference pressure response was measured in each of the other monitoring wells. This provided an overall $T$ value ($T_{\text{Effective}}$) for the connection between the entire length of the LAR-4 borehole and the entire length of each of the monitoring well boreholes; LAR-3, LAR-2, and LAR-1.
Figure 3.2 Idealized modified line search method using LAR-4 as the test well. For the initial test, a single packer was placed inside the casing of each well to obtain an effective $T$ value for the connection to each of the monitoring wells; LAR-3, LAR-2, and LAR-1.

Using the fracture locations identified during the initial downhole camera survey as a guide, the single packer in each of the monitoring wells was relocated to divide the number of fractures in each borehole by approximately half. The packer in LAR-4 (source well) remained inside the casing so the entire borehole was pressurized. A second slug-in test was conducted in LAR-4, and the interference pressure response from the bottom half of each monitoring well below the single packer was recorded (Figure 3.3). This provided $T$ values for the connection from the test well (LAR-4) to the
lower half of the fractures \(T_{\text{Lower Half}}\) in each of the monitoring wells; LAR-3, LAR-2, and LAR-1.

![Diagram of LAR wells with depth and test interval marked](image)

**Figure 3.3** Idealized modified line search method using LAR-4 as the test well. For the second test, the single packer in each of the monitoring wells is moved to divide the number of fractures or length of borehole by approximately half. The single packer in the test well, LAR-4, remained inside the casing so the entire open interval was pressurized.

The \(T\) values for the upper half of the fractures in each monitoring well \(T_{\text{Upper Half}}\) were estimated by subtracting the \(T_{\text{Lower Half}}\) value from the \(T_{\text{Effective}}\) value. This assumes that Transmissivity values are additive, as in the case of an idealized aquifer consisting of horizontal layers. The upper and lower half \(T\) values are compared, and the half with the higher \(T\) value is divided in half again by moving the single packer (Figure 3.4). In
cases where the $T_{\text{Lower Half}}$ and $T_{\text{Upper Half}}$ values are similar, both the upper and lower halves of the wellbores were divided.

![Diagram showing wellbores and test intervals](image)

**Figure 3.4** Idealized modified line search method using LAR-4 as the test well. For the third test, the $T$ value from the upper half of each monitoring well and lower half are compared, and the section with the highest $T$ value is split in half again using the single packer, and the slug test is repeated. This process is repeated until the desired resolution is obtained.

Each of these intervals were again divided and the slug tests repeated. Upon completion, $T$ values for 100% of the borehole, 75% of the borehole, the bottom 50% of the borehole, and the bottom 25% of the borehole were obtained. The additive $T$ property was assumed, and the effective $T$ for each quarter was calculated by subtraction (Figure 3.5).
The line search method was continued in this fashion until the desired resolution was obtained. The process was then repeated using a different borehole as the test well, i.e., LAR-2 as the test well and LAR-4, LAR-3, and LAR-1 as monitoring wells.

Figure 3.5  The additive $T$ property was assumed, and the effective $T$ for each quarter was calculated by subtraction.

3.4.2 Field Results

The slug tests using the MLS method were carried out to quarter-borehole resolution. In most cases, the $T_{\text{Upper Half}}$ and $T_{\text{Lower Half}}$ values were similar, so both the upper and lower half sections were divided. Three series of tests were carried out with LAR-4, LAR-2, and LAR-1 each being used as a test well. The interference pressure
responses in the monitoring wells were interpreted using the KGS model to calculate the
$T$ value for each test. Storativity values were ignored during evaluation of the modified
line search data, but they were evaluated during later straddle packer testing.

Upon completion of three rounds of testing where three separate wells were
used as the source well, $T$ values were calculated for each quarter borehole by
subtraction. The $T$ values were normalized by the length of the open interval in the
monitoring well to provide effective $K$ values for the connections. This was done to
account for the differences in lengths of open boreholes. $K$ values for each quarter
interval are shown on Figure 3.6.

The locations of the $K$ values on Figure 3.6 indicate the conductivity of that
quarter interval from an interference slug test in the entire length of the test well
borehole in that direction. For example, the values located along the right hand side of
LAR-1 on Figure 3.6 are the resulting conductivity values from the entire length of the
LAR-2 borehole. Subsequently, the $K$ values located along the left hand side of LAR-2
are the conductivity values from the entire length of the LAR-1 borehole. As mentioned
previously, the LAR-3 borehole was not utilized as a test well. Therefore, the $K$ values
listed to the right of the LAR-4 borehole were resultant of a slug test initiated in the LAR-
1 borehole and $K$ values to the left of the LAR-3 borehole were resultant from a slug test
initiated in the LAR-4 borehole.

$K$ values ranged from -0.25 to 0.43 m/day. Physically unrealistic negative $K$
values were obtained for some intervals. This occurred because when an interval along
the borehole is split by the straddle packer, only $T$ for the underlying sub-interval can be determined. $T$ for the upper sub-interval is determined by subtracting $T$ for the lower sub-interval from $T$ for the entire borehole. Negative values are interpreted to mean that the upper sub-interval has negligible contribution to $T$ of the entire borehole, so the negative values are set to 0.
Figure 3.6  $K$ values in m/day for connections between boreholes after completion of three rounds of testing where three separate wells (LAR-4, LAR-2, and LAR-1) were used as the test well. The location of the $K$ value on the figure indicates the conductivity of that quarter interval resulting from an interference slug test in the entire length of the neighboring test well borehole. For example, the values located along the right hand side of LAR-1 are the resulting conductivity values from the entire length of the LAR-2 borehole. Subsequently, the $K$ values located along the left hand side of LAR-2 are the conductivity values from the entire length of the LAR-1 borehole. $K$ values obtained between LAR-4 and LAR-2 are off-set in the background for clarity.
3.4.3 Interpretation

For interpretation of the MLS method results, the $K$ value of each quarter borehole as depicted in Figure 3.6 was assigned a relative value. $K < 0$ m/day was designated as Low (red), $0 < K < 0.15$ m/day was termed moderate (yellow), and $K > 0.15$ m/day was termed high $K$ (green) (Figure 3.7). This classification scheme allows the relative strengths of the connections between wellbores to be identified.

Connections were drawn between borehole quarters with similar conductivity classifications based on the line search method results (Figure 3.8). Relative connections between wells are represented in Figure 3.8 by bands of relative high $K$ (colored green), medium $K$ (colored yellow), and low $K$ (colored red). Two roughly flat-lying transmissive zones were identified in all four monitoring wells with the MLS method at depths ranging from approximately 32 m to 40 m and the other at depths ranging from 40 m to 60 m. Other connections were identified at shallower depths between pairs of wellbores, however, the connections did not appear to be continuous between all monitoring wells based upon the MLS method data.
Figure 3.7 Average $K$ values divided into 3 bins based on relative magnitude. $K < 0$ m/day was designated as Low (red), $0 < K < 0.15$ m/day was termed moderate (yellow), and $K > 0.15$ m/day was termed high $K$. 
Figure 3.8 Connections drawn between borehole quarters with similar conductivity classifications based on the line search method results. Colored bands correspond to high $K$ (colored green), medium $K$ (colored yellow), and low $K$ (colored red).

3.5 STRADDLE PACKER TESTING

The MLS method appears to be a viable approach to identifying sections of a borehole with higher conductivity and greater connection between monitoring wells, but lacks the precision necessary to characterize individual fractures or fracture sets. To
accomplish this, straddle packers were incorporated into the testing regime to evaluate each of the moderate to high $K$ zones identified in the LAR-4 borehole.

3.5.1 Methodology

A pair of inflatable straddle packers was used in each well to isolate individual fractures or fracture sets and measure the hydraulic response from a slug test initiated in the test well, LAR-4. The packers used in the source well were RocTest LP 102-190, with a rubber gland that contacts a length of the borehole spanning approximately 0.7 m. The isolated interval between the packers spanned 3 m from the top of the lower packer to the bottom of the upper packer. The packers were inflated with N$_2$ gas to approximately 1.2 MPa (180 psi) above hydrostatic pressure. Low-pressure packers [Svenson et al. 2005] were used in the monitoring wells and were inflated to approximately 0.2 MPa (29 psi) above hydrostatic pressure. The low-pressure packers contacted the borehole wall over approximately 0.4 m and were separated by a distance of 2.4 m.

The pressure responses from the slug tests were monitored using a pressure transducer in the test well and in each of the monitoring wells. A Wika Submersible Liquid Level Transmitter (Model LS-10) with a span of 50 psi was used to monitor pressure in the test well (LAR-4) and one with a 15 psi span was used in monitoring well LAR-3. Accuracy of the Wika transducers is approximately 1.0% of full scale, according to the manufacturer. Pressure transducers in monitoring wells LAR-2 and LAR-1 were
Honeywell Wet/Wet Differential Pressure Sensors (model PX26) with spans of 5 psi and 1 psi, respectively. The Honeywell transducers were configured with operational amplifiers and then encapsulated in epoxy for deployment down-hole. Amplifying and calibrating the Honeywell transducers yielded accuracies of approximately 0.1% of the full scale. Each of the transducers used a 0-5 VDC output, which was interfaced with a 24-bit analog-digital converter and the resulting signal was recorded by a National Instruments data acquisition system at a rate of 1 HZ.

A well head assembly with a three-way solenoid valve was used at the source well to control air flow into the head-space above the water in the 2-inch PVC pipe string. To conduct a slug test, the solenoid valve was opened to a tank of pressurized air attached to a pressure regulator. Air flowed into the head-space above the water level inside the PVC pipe string and the pressure increased over a period of approximately 8 seconds. This caused an abrupt increase in pressure at the level of the fracture, and is equivalent to dropping a weighted slug into the well during a conventional slug-in test. The water level in the PVC pipe string was lowered in response to the pressurized head space, and this was detected by the submersible transducer. Eventually, the water level in the pipe string dropped by an amount equal to the head of the pressure in the tank and the system was equilibrated. The solenoid valve was then vented to the atmosphere, which rapidly dropped the pressure inside the pipe string and initiated a slug-out test. The pressure dropped over 5-6 seconds during the slug-out test, several
seconds faster than pressurization during the slug-in. The solenoid valve was attached to an electric timer so that slug tests could be repeated automatically.

3.5.2 RESULTS

Interference slug tests were conducted between conductive intervals in LAR-4 and other locations in LAR-1, LAR-2, and LAR-3. Three conductive intervals were identified in LAR-4 using the MLS method and are known from previous work (Svenson, 2009) (Figure 2.4), and they will be designated Zone I (26 m depth), Zone II (35 m depth) and Zone III (47 m depth). Zone 1 of LAR-4 was indicated as having only a moderate $K$ during the MLS method, and appears to be discontinuous.

Observation intervals at the monitoring wells will be designated by the well number and a letter. For example, there are 5 depth zones in LAR-3 and they are designated as $a$ through $e$ (Figure 3.9). Each interference test will be designated by specifying the locations where the pressure originated and the interval where it was detected. For example, a test originating in Zone II in LAR-4 and detected at depth zone $c$ in LAR-3 would be: LAR-4 Zone II $\rightarrow$ LAR-3$c$. This will be abbreviated as: 4II-3$c$.

Analysis using KGS Model

The straddle packer tests were analyzed by fitting the pressure response data using the KGS model implemented in Aqtesolve. The KGS model was developed by the Kansas Geological Society (Butler et al. 1993; Hyder et al. 1994; Butler 1995; Liu and
Butler 1995] as a semi-analytical method for interpretation of interference slug tests in both confined and unconfined aquifers. The model accounts for elastic storativity and anisotropy, wellbore storage in the source well, as well as partial penetration of both the source and the monitoring well. We chose to use the KGS model for interpretation of the interference data primarily because of its ability to accommodate partially penetrating wells at both the source and monitoring wells. For the model set up, a uniform aquifer thickness of 53 m was used, and the borehole span between straddle packers was considered equivalent to the screen length. The KGS model allowed for the screened intervals (or intervals isolated by straddle packers) to be located at various depths at each well.

The initial response to many of the slug-in tests was a pressure drop at the monitoring well as a result of hydromechanical effects outlined in Chapter 2. The KGS model was only capable of predicting pressure increases during slug-in tests, so the early decreases in pressure were ignored. The results from LAR-4 to each of the monitoring wells are described below.

*Interference Slug Test: LAR-4 to LAR-3*

The KGS model fit observed pressures at both the source well (LAR-4) and the monitoring well for test 4I-3a, giving $K = 3.69\times10^{-6}$ m/sec and $Ss = 3.10\times10^{-6}$ sec$^{-1}$. In test 4I-3b, however, the KGS model could fit either the source well or the observation well, but
not both simultaneously (Figure 3.9). In this case, the fit to the monitoring well data was given priority, and this produced $K = 4.96e-6$ m/sec and $Ss = 3.54e-6$ sec$^{-1}$.

The monitoring well data were given priority in the parameter estimate because they more strongly reflect the properties between the source and the observation wells. In contrast, the response at the source well could be influenced by formation properties outside of the region between the source and observation wells. For example, a fracture that intersected Zone I in LAR-4 and extended away from LAR-3 could have caused the response in the source well to decrease more rapidly than would be expected if LAR-4 was enveloped by uniform material.
Figure 3.9: Pressure as a function of time during interference slug tests between Zone 1 of the source well, LAR-4 (solid circles) and monitoring well LAR-3 (open circles). The KGS analytical model fit to the field data is represented with solid lines.

Due to the high density of fractures within the LAR-4 borehole at approximately 32 meters depth, the source well was divided into three sub-zones, designated Zone IIa, IIb, and IIc. The KGS analytical model was able to fit the interference pressure response for both 4IIa-3b and 4IIa-3c, although the rate of decay of the test well pressure response was overestimated above the field data for both tests (Figure 3.10). A $K$ value of $5.15e^{-6}$ m/sec and $S_s$ value of $1.31e^{-6}$ sec$^{-1}$ were calculated for 4IIa-3b, and a $K$ value of $1.18e^{-6}$ m/sec and $S_s$ value of $1.93e^{-7}$ sec$^{-1}$ were calculated for 4IIa-3c. The interference pressure response in 4IIa-3d was not discernable above background noise.
Figure 3.10: Interference slug test pressure response from zone IIa of test well (LAR-4) to monitoring well LAR-3.

The KGS analytical model results for 4IIb were similar to the 4IIa test analyses, with the exception of 4IIb-3c where the decay rate of the test well pressure is greatly overestimated (Figure 3.11). Like Zone 4IIa, the interference pressure response in 4IIb-
3d was not discernable above background noise. A $K$ value of $8.36 \times 10^{-6}$ m/sec and $S_s$ value of $3.63 \times 10^{-7}$ sec$^{-1}$ were calculated for 4Ilb-3b. The analysis of the pressures in 4Ilb-3c yielded a $K$ value of $8.36 \times 10^{-7}$ m/sec and $S_s$ value of $9.69 \times 10^{-8}$ sec$^{-1}$.

\[\begin{align*}
4\text{Ilb-3b} \\
\text{LAR-4} \\
\end{align*}\]

\[\begin{align*}
4\text{Ilb-3c} \\
\text{LAR-3} \\
\end{align*}\]

\[\begin{align*}
4\text{Ilb-3d} \\
\end{align*}\]

**Figure 3.11:** Interference slug test pressure response from zone IIb of test well LAR-4 to monitoring well LAR-3.
Figure 3.12: Interference slug test pressure response from zone IIc of test well LAR-4 to monitoring well LAR-3.

The KGS analytical model results for 4IIc-3b yielded a $K = 4.60e-6$ m/sec and $Ss = 8.63e-6$ sec$^{-1}$ and a $K = 3.48e-6$ m/sec and $Ss = 7.62e-5$ sec$^{-1}$ for 4IIc-3c (Figure 3.12). The interference pressure response in 4IIc-3d was undetectable. Although the $K$ values for
the 4IIc zone were similar to both zones 4IIa and 4IIb, the $S_S$ values for zone 4IIc were nearly two orders of magnitude larger.

A $K$ value of $2.40 \times 10^{-6}$ m/sec and $S_S = 4.33 \times 10^{-7}$ sec$^{-1}$ were calculated for 4III-3d (Figure 3.13). The analysis of the pressures in 4III-3e yielded a $K = 1.00 \times 10^{-5}$ m/sec and $S_S = 1.02 \times 10^{-6}$ sec$^{-1}$. The KGS model was able to fit both the 4III-3e test well and monitoring well response well.

**Figure 3.13:** Interference slug test pressure response from zone III of test well LAR-4 to monitoring well LAR-3.
The conductivity values for all straddle packer tests conducted between LAR-4 and monitoring well LAR-3 ranged from 8.36e-7 m/sec in 4IIb-3c to 1.00e-5 m/sec in 4III-3e. Specific storage values ranged from 9.69e-8 sec\(^{-1}\) in 4IIb-3c to 7.26e-5 sec\(^{-1}\) in 4IIc-3c (Table 3.1). Figure 3.14 presents conductivity and specific storage values along connections between LAR-4 and LAR-3 as vectors. Each vector is coded to represent varying degrees of \(K\) and \(S_s\) as follows: low \(K < 3.0e-6\) m/sec (dotted vector), moderate \(K > 3.0e-6\) m/sec but \(< 6.0e-6\) m/sec (dashed vector), high \(K > 6.0e-6\) m/sec (solid vector); low \(S_s < 6.0e-6\) sec\(^{-1}\) (purple vector), moderate \(S_s > 6.0e-6\) sec\(^{-1}\) but \(< 3.0e-5\) sec\(^{-1}\) (blue vector), and high \(S_s > 3.0e-5\) sec\(^{-1}\) (red vector). Solid black vectors indicate no detectable connection above the background noise of the transducers. Background colors correspond to the low (red), moderate (yellow), and high (green) relative \(K\) connections between wellbores identified during the MLS method.
Figure 3.14: Relative $K$ and $S_s$ values along connections between LAR-4 and LAR-3. Background colors correspond to low (red), moderate (yellow), and high (green) relative conductivity connections between wellbores identified during the MLS method. Vectors showing the connections between wellbores are coded to symbolize varying degrees of $K$ and $S_s$ as follows: low $K < 3.0e-6$ m/sec (dotted vector), moderate $K > 3.0e-6$ m/sec but $< 6.0e-6$ m/sec (dashed vector), high $K > 6.0e-6$ m/sec (solid vector); low $S_s < 6.0e-6$ sec$^{-1}$ (purple vector), moderate $S_s > 6.0e-6$ sec$^{-1}$ but $< 3.0e-5$ sec$^{-1}$ (blue vector), and high $S_s > 3.0e-5$ sec$^{-1}$ (red vector).

**Interference Slug Test: LAR-4 to LAR-2 and LAR-4 to LAR-1**

The KGS analytical model was able to fit the interference pressure response in each of the zones for both LAR-2 and LAR-1. The KGS model was able to fit both the test
well and LAR-2 monitoring well in tests 4I-2b and 4III-2i, but overestimated the LAR-4 test well response in tests 4I-2c, 4IIa-2d, 4IIa-2e, 4IIa-2f, 4IIb-2f, and 4III-2h, and underestimated the test well response in tests 4IIc-2e and 4IIc-2f (Figures 3.15 through 3.19). Calculated $K$ values range from 1.15e-6 m/sec for 4IIb-2f to 1.05e-5 m/sec for 4III-2i, whereas $S_s$ values range from 1.02e-6 sec$^{-1}$ for 4III-2i to 3.86e-5 sec$^{-1}$ for 4III-2h (Table 3.1).

For LAR-1, the KGs model was able to fit the interference pressure response peak arrival time and magnitude in all of the zones. However, the model could not fit the overall shape of the interference pressure response in test 4IIa-1b, 4IIb-1b, and 4III-1b. In general, the KGs model was able to fit both the test well and LAR-1 monitoring well in tests 4I-1a, 4IIa-1b, and 4III-1c, overestimated the LAR-4 test well response in tests 4I-1b, 4IIa-1a, and 4IIb-1a, while underestimating the test well response in tests 4IIa-1c, 4IIc-1a, 4IIb-1b, 4IIb-1c, 4III-1b, and 4IIc-1b (Figures 3.20 through 3.24). Calculated $K$ values range from 2.42e-6 m/sec for 4I-1a to 4.20e-5 m/sec for 4IIb-1c, whereas values of $S_s$ range from 1.29e-6 sec$^{-1}$ for 4III-1b to 5.45e-5 sec$^{-1}$ for 4IIc-1a (Table 3.1).
Figure 3.15: Interference slug test pressure response from Zone I of test well LAR-4 to monitoring well LAR-2.
Figure 3.16: Interference slug test pressure response from Zone IIa of test well LAR-4 to monitoring well LAR-2.
Figure 3.17: Interference slug test pressure response from Zone IIb of test well LAR-4 to monitoring well LAR-2.
Figure 3.18: Interference slug test pressure response from Zone IIc test well LAR-4 to monitoring well LAR-2.
Figure 3.19: Interference slug test pressure response from Zone IIc test well LAR-4 to monitoring well LAR-2.
Figure 3.20: Interference slug test pressure response from Zone I test well LAR-4 to monitoring well LAR-1.
Figure 3.21: Interference slug test pressure response from Zone Iia test well LAR-4 to monitoring well LAR-1.
Figure 3.22: Interference slug test pressure response from Zone IIb test well LAR-4 to monitoring well LAR-1.
Figure 3.23: Interference slug test pressure response from Zone llc test well LAR-4 to monitoring well LAR-1.
Figure 3.24: Interference slug test pressure response from Zone III test well LAR-4 to monitoring well LAR-1.
Figure 3.25: Relative $K$ and $Ss$ values along connections between LAR-4 and LAR-3. Background colors correspond to low (red), moderate (yellow), and high (green) relative conductivity connections between wellbores identified during the MLS method. Vectors showing the connections between wellbores are coded to symbolize varying degrees of $K$ and $Ss$ as follows: low $K < 3.0e-6$ m/sec (dotted vector), moderate $K > 3.0e-6$ m/sec but $< 6.0e-6$ m/sec (dashed vector), high $K > 6.0e-6$ m/sec (solid vector); low $Ss < 6.0e-6$ sec$^{-1}$ (purple vector), moderate $Ss > 6.0e-6$ sec$^{-1}$ but $< 3.0e-5$ sec$^{-1}$ (blue vector), and high $Ss > 3.0e-5$ sec$^{-1}$ (red vector).
Figure 3.26: Relative $K$ and $S_s$ values along connections between LAR-4 and LAR-3. Background colors correspond to low (red), moderate (yellow), and high (green) relative conductivity connections between wellbores identified during the MLS method. Vectors showing the connections between wellbores are coded to symbolize varying degrees of $K$ and $S_s$ as follows: low $K < 3.0 \times 10^{-6}$ m/sec (dotted vector), moderate $K > 3.0 \times 10^{-6}$ m/sec but $< 6.0 \times 10^{-6}$ m/sec (dashed vector), high $K > 6.0 \times 10^{-6}$ m/sec (solid vector); low $S_s < 6.0 \times 10^{-6}$ sec$^{-1}$ (purple vector), moderate $S_s > 6.0 \times 10^{-6}$ sec$^{-1}$ but $< 3.0 \times 10^{-5}$ sec$^{-1}$ (blue vector), and high $S_s > 3.0 \times 10^{-5}$ sec$^{-1}$ (red vector).
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3.6 Discussion

The KGS analytical model was used to analyze straddle packer interference slug tests at the Clemson University lower well field, and in all cases where the pressure response was detectable above background noise the model was able to predict the magnitude and arrival time of the interference response. However, the KGS analytical model was unable to predict the early time reverse water level discussed in Chapter II, and was not always able to replicate the overall shape of the interference pressure response. Additionally, the KGS model often overestimated or underestimated the pressure signal decay rate of the test well. This inability to match the source well response is likely a combination of the complexity of the fracture network extending into the formation and localized phenomena such as well skin near the test well.

The KGS analysis of the straddle packer field data indicate $K$ values ranging from 8.36e-7 m/sec to 4.20e-5 m/sec whereas $S_s$ values ranged from 9.69e-8 sec$^{-1}$ to 7.26e-5 sec$^{-1}$. Results for the interference slug tests are similar to ranges of values identified by Svenson [2006] during single well straddle packer slug tests of the LAR-4 borehole.

Figures 3.14, 3.25, and 3.26 indicate the relative $K$ and $S_s$ values from each of the zones or subzones in LAR-4 to monitoring wells LAR-3, LAR-2, and LAR-1, respectively. For LAR-4 and LAR-3, the results of the straddle packer testing confirm the preliminary findings of the interference slug test line search method for the lower zone (4III) and middle zone (4II), with the highest $K$ values at 4IIb-3b and 4III-3e. Some ambiguity exists for connections between zones 4IIa, 4IIb, and 4IIc and LAR-3 however, and is likely due
to the close proximity of the fractures. These connections are complexly interconnected so that multiple high $K$ flow paths exist. Moderate $K$ connections from the upper LAR-4 zone which were not identified between LAR-4 and LAR-3 with the MLS Method were identified with the straddle packer testing between the upper LAR-4 zone and subzones 3a and 3b.

For LAR-4 and LAR-2, the straddle packer testing results generally agree with the MLS method results. Straddle packer test results indicate that LAR-4 and LAR-2 are connected along relatively flat lying fracture planes, with the connection between 4III-2i having the highest $K$ value. Moderate $K$ connections were identified between 4IIa-2d, 4IIa-2f, 4IIc-2f, and 4I-2b. Like the LAR-4 and LAR-3 straddle packer test results, the moderate $K$ connection from the upper LAR-4 zone (4I) was not identified during the MLS Method.

For LAR-4 and LAR-1, the straddle packer testing indicates that LAR-1 zone b and zone c are most hydraulically connected with all of the tested zones within LAR-4. Like LAR-2, some ambiguity exists between the flow paths. Straddle packer test results also seem to indicate a steeply dipping connection between the LAR-4 upper interval (4I) and the lower LAR-1 subzones b and c.

Based upon interpretation of the straddle packer and MLS method data, the hydraulically dominant fracture connections within the Clemson University lower well field were drawn between wellbores (Figure 3.27).
Figure 3.27  Hydraulically dominant fractures or fracture zones identified at the Clemson University lower well field based upon interpretation of straddle packer and MLS method data.

All of the data indicate a single flat-lying fracture that connects all of the wellbores at approximately 47 m. Additionally, a set of several interconnected, roughly horizontal fractures create an approximately 4 m thick fractured zone of moderate to high $K$ that connects all of the wellbores in the lower well field at approximately 35 m. The upper fractured interval identified by Svenson [2006] at 26 m appears to be poorly connected to the other boreholes compared to the zones at 35 m and 47 m. This is interesting because the transmissivity of the upper fracture zone in LAR-4 is relatively
large, according to Svenson (2006). Data obtained from the LAR-1 straddle packer tests seem to indicate a steeply dipping, relatively moderate $K$ connection between the upper fracture interval in LAR-4 and the lower fractures in LAR-1. However, caution must be exercised in this interpretation. It is possible that rather than a steeply dipping connection between the upper zone in LAR-4 and the lower fracture intervals in LAR-1, the interference slug test pressure pulse may be circumventing the isolated interval in LAR-3 and traveling down the LAR-3 borehole into the more compliant and conductive fractures connecting all of the wellbores at 35 and 47 m.

The MLS method was able to greatly reduce the number of tests required to identify the most hydraulically active intervals in each of the boreholes at the Clemson University lower well field. Approximately 12 slug tests were required to carry the MLS method out to a quarter borehole resolution, and the MLS method was able to subsequently identify two flat lying fracture zones connecting the lower halves of each of the boreholes and several shallower, discontinuous connections between pairs of wellbores. Generally, straddle packer testing confirmed the preliminary findings of the MLS method, and were able to provide further insight into the hydraulic connections between wellbores.

### 3.7 Conclusion

Permeable connections between boreholes are an important aspect of the hydrogeology of fractured rock and identifying these connections can require testing
many possible connections with straddle packers. A method was evaluated for identifying permeable connections between boreholes and was designed to reduce the number of packer tests by searching of progressively narrower intervals. The approach begins by determining effective T using slug tests between open boreholes. The open intervals are bisected by individual packers, and the slug tests are repeated. Transmissivities are calculated for the interval underlying the packer, and T for the overlying interval is determined by subtraction. This process is repeated by progressively bisecting the most transmissive intervals. This approach resembles the line searching technique used to find minima in a 1-D vector.

A field application of the technique identified conductive intervals at 35 m and 47 m depth at a site in western South Carolina underlain by fractured biotite gneiss. These intervals were tested in more detail using slug tests between straddle packers, which allowed the hydraulic properties of along specific connections to be determined.
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