PATTLE DELAMORE PARTNERS LTD
RAGLAN RAPID INFILTRATION INVESTIGATION REPORT

Raglan Rapid Infiltration Investigation Report

Prepared for
Waikato District Council

January 2002

pdp

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Quality Control Sheet

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Limitations:

The report has been prepared for Waikato District Council, according to their instructions, for the particular objectives described in the report. The information contained in the report should not be used by anyone else or for any other purposes.

Executive Summary

A field study involving groundwater investigations, drilling and computer modelling has been undertaken for Waikato District Council (WDC) to investigate the land disposal option of rapid infiltration (RI) for disposal of treated sewage from the Raglan municipal sewage treatment plant (STP). A potential RI site in sand dunes near the surf club at the northeastern end of Ngarunui Beach has been investigated. Potential sites for RI were identified in a feasibility study for several different land treatment options by Pattle Delamore Partners Limited (PDP) (June 2001).

This study has concluded that effluent disposal by RI at a rate of 11 m^3/d/100 m of coastline can be achieved. Therefore, based on a future predicted average sewage flow of 1,100m^3/d, there is insufficient length of suitable coastline available for a stand-alone RI system.

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1.0 Introduction

Pattle Delamore Partners Ltd (PDP) has been commissioned by Waikato District Council (WDC) to undertake an investigation into the use of rapid infiltration (RI) for disposal of treated sewage effluent from the Raglan municipal sewage treatment plant (STP). This work follows a preliminary feasibility report by PDP (June 2001) that evaluated several land treatment options. The June 2001 report recommended that the RI option be investigated in more detail.

The project, as with the previous June 2001 investigation, is being undertaken for WDC in consultation with the Raglan Wastewater Working Group (Working Group) which together are currently considering land treatment and disposal of effluent as an alternative to the current ocean outfall system. Prior to this site investigation, discussions between WDC and members of the Working Group identified the Raglan Surf Lifesaving Club site as the favoured RI site from among three areas identified in the June 2001 report (PDP, 2001). Rapid infiltration basins would be positioned behind the foredunes in this area.

The project 'brief' for this RI study required investigating the potential RI area using drilling investigations, installation of monitoring wells to determine groundwater levels, monitoring of water levels over a two week period, and computer modelling of the results of the above to simulate the effects of rapid infiltration on water table conditions. Treated effluent disposed into the sand dune area via RI basins will raise the level of the groundwater table (see Section 1.1 below). The computer modelling takes various parameters of sand hydraulic conductivity, aquifer geometry and geology and existing water levels into account to determine the potential loading rates available.

1.1 Rapid Infiltration Systems

RI is the process whereby effluent infiltrates vertically into the ground when applied to basins formed in soils with high hydraulic conductivity (such as sands). The basins are periodically flooded and this cycle of loading and resting restores aerobic conditions in the soil and can help improve nitrogen removal from the effluent. Effluent percolates to the groundwater and eventually flows to a surface water body. The land area required for disposal of an equivalent volume of effluent by RI is much smaller than for disposal by slow rate irrigation (SRI) and RI is not as adversely affected by antecedent weather conditions. There is a RI system operated by Thames Coromandel District Council at Pauanui which has been successfully disposing of effluent for over 20 years.

For the Raglan system, each RI basin would be about 3 m wide by 9 m long with a depth of 1 m. The basins would be spaced in clusters along the sand dunes parallel with the beach. Locked wooden lids would be constructed over the basins to prevent entry by the public. The basins would not be visible from the beach and as such would not be visually intrusive. As discussed in the June 2001 report, the effluent disposed of to the RI basins would be of high quality to minimise the potential for any adverse environmental effects of public health concerns.

2.0 Field Work

The site was visited by PDP over a three day period from 21 to 23 November 2001. During this time a drilling investigation was undertaken, with the installation of four monitoring wells along a cross section through the sand dune area to the south west of the Raglan Surf Lifesaving Club buildings (Figure 1 and 2). The bores were drilled using a tractor mounted auger with wire-line coring. Bores MW1, MW3 and MW4 were drilled into the underlying rock/low permeability strata. MW2 was finished at the base of the sand layer. Logs of the bores are appended (Appendix A). The bores were completed with PVC casing and slotted screen, with a locked steel above-ground toby box set in a concrete pad at ground surface. The monitoring well screens were set at the base of the sand aquifer (see borelogs in Appendix A for details). A further borehole was drilled on the beach at the toe of the sand dunes approximately 200 m northeast of the beach access boardwalk onto Ngarunui Beach from the Wainui Reserve, further southwest from the Raglan Surf Lifesaving Club site. This borehole contacted assumed low permeability materials/rock at 4.35 m depth below ground level, indicating a similar sand thickness at this end of the beach compared to the Raglan Surf Lifesaving Club site.

After the monitoring wells had been installed a series of 'slug tests' were undertaken to gather information to allow hydraulic conductivity calculations to be made for the sand aquifer materials. Slug tests were undertaken on MW1 and MW3 on 23 November 2001, and on MW2 on 6 December 2001. Results of the analysis of these tests using the pump test analysis software AquiferWin32 and the Bouwer and Rice method are appended (Appendix B).

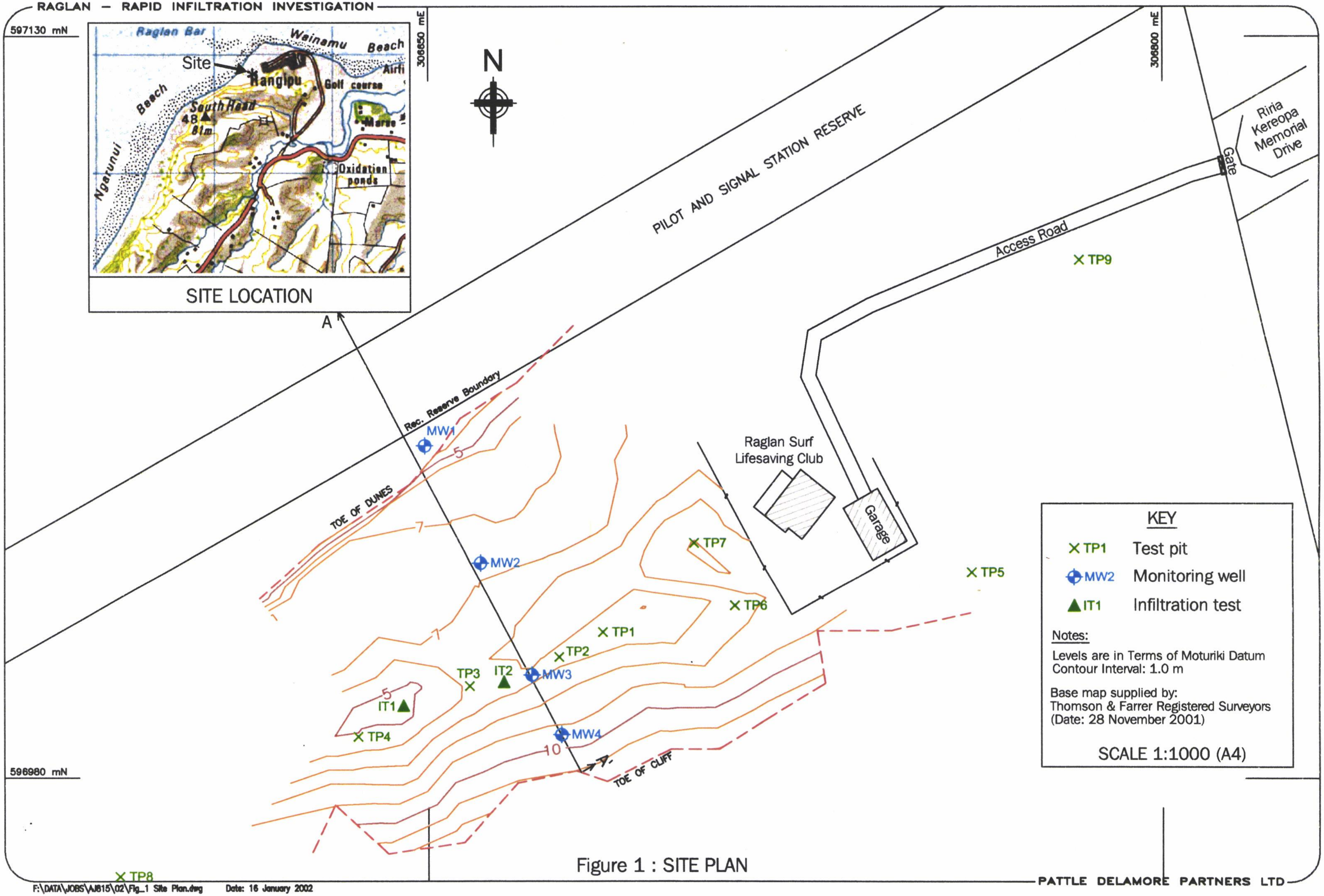
Water level transducers with data loggers were installed in MW1, MW2 and MW3, with an additional transducer in MW3 above the water table used to collect atmospheric pressure data to allow compensation of water level recordings to be undertaken. The loggers were started on 23 November 2001 and left in place until 6 December 2001 when they were removed and the water level data was downloaded. Water levels during the logged period are presented in Appendix C.

The monitoring wells were all measured with a water dipper to check static water levels on 6 December 2001 during a separate site visit made to retrieve the data loggers.

Test pits were dug using a mechanical excavator across the area (TP1 to TP9 on Figure 1) to check visually for sand thickness and any obvious layers, and to allow inspection by WDC's coastal erosion consultant (Mr Jim Dahm) for details regarding historic coastal processes in the area. All the test pits did not encounter materials at depth other than sand, except for pit TP5 which encountered a clayey silt at 1.9 m depth. Pits TP1, TP3, TP8 and TP9 all extended to between 4.0 m and 4.6 m deep, and the other shallower pits mostly to between 2.0 m and 2.5 m deep. The test pits were located in the approximate position where any rapid infiltration basins would likely be constructed.

Infiltration tests were carried out at two locations (IT1 and IT2 on Figure 1) using a double ring infiltrometer. The results of these tests are presented in Appendix D.

RAGLAN – RAPID INFILTRATION INVESTIGATION



KEY

× TP1	Test pit
⊕ MW2	Monitoring well
▲ IT1	Infiltration test

Notes:

Levels are in Terms of Moturiki Datum
Contour Interval: 1.0 m

Base map supplied by:
Thomson & Farrer Registered Surveyors
(Date: 28 November 2001)

SCALE 1:1000 (A4)

Figure 1 : SITE PLAN

PATTLE DELAMORE PARTNERS LTD

F:\DATA\JOBS\AJ815\02\Fig_1 Site Plan.dwg Date: 16 January 2002

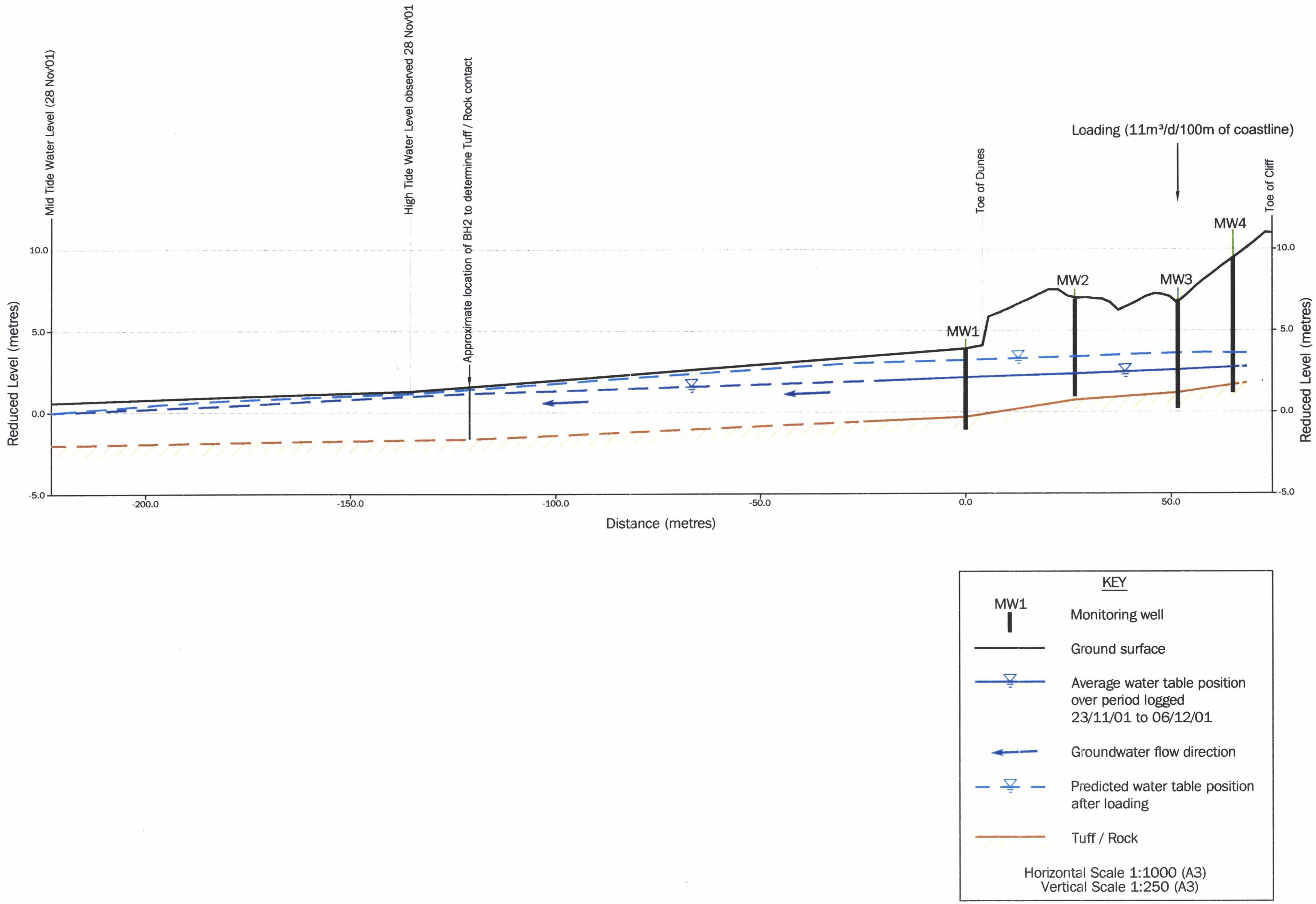


Figure 2 : CROSS-SECTION A-A'

A sand sample was taken from site IT1 at 700 to 800 mm depth (Figure 1) during the field visit. The sample was sent to a geotechnical laboratory and a dry grading was undertaken (Appendix E).

A survey was undertaken over the area of investigation by Thompson and Farrer Limited to define the topography and measure the levels of the monitoring wells in relation to mean sea level.

3.0 Hydrogeology

The area investigated near the Raglan Surf Lifesaving Club at the northeastern end of Ngarunui Beach is comprised of sand dunes and beach sand deposits, which in turn overlie andesite and basalt rocks of the Alexandra Volcanics (Kear, 1960). The sand dunes and beach sand deposits in the area behind the fore-dune where the potential RI system would be located are mostly around 6 m thick (MW2 and MW3) and up to approximately 8 m near MW4. Borehole MW1 at the toe of the dunes on the beach encountered 4.1 m of sand before a thin layer of silty material (approximately 200 mm) and then rock. Borehole MW2 was finished at the base of the sand layer, at 6.0 m depth, and was noted to be at the start of compact material (assumed to be the top of the low permeability layer/rock). Boreholes MW3 and MW4 were advanced past the sand layer into low permeability material containing tuff and rock fragments (see bore logs in Appendix A). A borehole was drilled to ascertain approximate depth to rock/low permeability material at a point approximately 12 m northwest of MW1 (close to the section line in Figure 2) encountered hard materials at 3.2 m depth. This borehole was not cored, sampled or surveyed. The above information indicates a relatively flat basement layer of rock/low permeability materials, sloping gently towards the sea, based on the limited drilling and surveyed levels.

The results of the infiltration testing show high saturated infiltration rates of 600 and 800 mm/hr, well above that required for RI operation. However, these results would need to be factored down to take account of various factors for a RI pit design, given they are for fresh water, on undisturbed sand without a history of previous RI operation. Despite this, at approximately half of the infiltration rates found, there would be sufficient capacity for a RI system to operate effectively. However, infiltration rate is not the only factor affecting performance of an RI system and groundwater mounding must also be considered (discussed in Section 5).

Groundwater movement within the dunes is towards the sea, with a gradient of 0.009 to 0.010 in the vicinity of MW1 to MW4, along the NNE-SSE cross-section through the wells (Figure 2). Results of the slug tests in wells MW1-3 indicate an hydraulic conductivity of about 8 x 10^{-5} m/s, i.e. geometric mean of slug tests (Table 1). Analysis of the grain size distribution for the sand sample taken from IT1 (Figure 1) at 700 – 800 mm depth (Appendix E) gives an approximate estimate of hydraulic conductivity at $1\text{x}10^{-4}$ m/s. The more accurate estimate provided by the slug test analysis was used for the groundwater modelling. The sand grading shows a fine sand with a D_{10} (10 percentile particle size) of approximately 0.13 mm. The sand is similarly graded to the sample tested from a site further southeast on Ngarunui Beach (Sample S3) as part of the previous PDP investigation (PDP, 2001).

Measurements on 6 December 2001 showed groundwater levels at 1.67 m below ground level at MW1, and greater than 4m below ground for MW2 to MW4. (Appendix C, Table C1). Level logger results showed relatively small level fluctuations in the four monitoring wells with levels generally dropping over the period, with a maximum variation of approximately 200 mm noted in MW1, likely the result of significant rainfall on 22 November 2001. (Rainfall record over the period is included in Appendix C, Table C2). No tidal fluctuations were noted in any of the water level logger records. The larger level fluctuations at MW1, and MW2 when compared to MW3, are likely a reflection of the depth to groundwater and the catchment topography and its effect on rainfall infiltration.

Table 1: Summary of Hydraulic Conductivity Testing

Hydrogeological Unit	**Horizontal K (m/s)**				**Vertical K (m/s)**^{3}
Unit 1 Top dune and marine sands	Slug Tests	MW1	MW2	MW3	Infiltration Tests$^{(1)}$: IT1: 2 x 10^{-4} m/s IT2: 2 x 10^{-4} m/s
	1	7 x 10^{-5}	7 x 10^{-5}	1 x 10^{-4}	
	2	7 x 10^{-5}	7 x 10^{-5}	1 x 10^{-4}	
	3	9 x 10^{-5}			
	Average:	8 x 10^{-5}	7 x 10^{-5}	1 x 10^{-4}	
	Geometric mean of slug test results: 8 x 10^{-5}				
	$^{(1)}$ $^{(2)}$Grain size analysis: 1x 10^{-4}				

Notes: 1. *Slug tests give more reliable results on hydraulic conductivity (K) than K calculated using grain size or infiltration tests. Hydraulic conductivity used in the model is based on slug test results (which are conservative).*

2. *Based on Hazen method (1911), K(cm/s)=C (d10)^2, where C= 60 (average) for fine sand.*

3. *Kv is likely to be slightly less than Kh (2 to 5 times) due to minor stratification from deposition. However its value has no effect on the model with one single layer.*

4.0 Existing Groundwater Regime

4.1 Conceptual Model

The field data from Section 2.0 has been used to develop an understanding of the groundwater behaviour beneath the disposal site. This information is used to define hydraulic properties of the sand layer aquifer as well as groundwater flow paths, rate and velocities. The hydrogeological section (Figure 2) shows the elements of the conceptual model. The key aspects of the conceptualisation are:

- A geological framework comprising about 4 m (at the beach) of a fine sand layer underlain by the andesite/basalt rocks of the Alexandra Volcanics Formation. The geometric mean of the hydraulic conductivity of the sand is 8 x 10^{-5} m/s (based on the slug tests results). In the PDP June 2001 study, prior to drilling investigations, it was assumed that the sand thickness was much greater, around 30m, based on available information for similar west coast sand dune formations in the upper North Island.

- Groundwater flow is from the hill, i.e., the exposed weathered rocks, towards the sea.
- Recharge to the groundwater system occurs primarily via rainfall infiltration.
- Recharge in the sand layer is expected to be approximately 30% of rainfall.
- Discharge from the groundwater flow system occurs via seepage to sea.

The following assumptions were used during conceptualisation. These assumptions should be taken into account when interpreting the results.

- The average water levels in the monitoring period were used in the model as the average long-term water levels, as no long-term monitoring data is available. It is not expected that long term water levels in the boreholes would be much different.
- It is assumed that the underlying volcanic rock forms an impermeable boundary for downward movement of infiltrated water (a conservative assumption). The permeability in this rock is variable and depends on the degree of fracturing and weathering effects. However, an approximately 200 mm thick silty layer has been identified at the top of the rock (e.g. see bore log for MW1). The silty layer is likely to prevent or greatly reduce the downward movement of the infiltrated water into the volcanic rocks.

4.1.1 Groundwater Flow Paths and Rates

The water levels recorded at the monitoring wells provide an indication of the groundwater flow directions at the site. Groundwater flow occurs from high piezometric level close to the exposed weathered rocks of the steep ground to the southeast, to low piezometric level at the beach at a rate which is dependent upon the hydraulic conductivity of the strata between the two points of measurement.

The hydraulic conductivity values from the slug tests performed on the monitoring wells, grain size analysis and infiltration tests are presented in Table 1. Using the slug test results (K = 8 x 10^{-5} m/s), slope of the impermeable bed (0.02), the hydraulic gradient and the modified Dupuit method for an unconfined aquifer with a sloping base, the flow rate through the beach is 0.28 m^3/d for 1 m width of coastline. This assumes that the average depth of sand is 2m at the mid-tide position.

4.1.2 Groundwater Velocity

Using the above flow rate (Q) and porosity of 0.25 (for fine sand, n) and cross sectional area (A) at the discharge point (2 m^2), the groundwater velocity (V) is 0.6 m/d (V=Q/nA) or 220 m/year. This velocity indicates that any water migrating through the RI loading zone (close to MW3, which is about 270 m from the position of average sea level) will take about 1 year to reach the sea.

4.1.3 Saltwater Interface

It is expected the aquifer will contain saline water below the high tide level. Above this level brackish conditions will occur in the zone of mixing, with down beach movement of fresh groundwater recharge.

4.1.4 Water Balance

The water balance for the groundwater flow beneath the proposed wastewater disposal site is relatively simple, comprising recharge from infiltrating rainwater and discharge to the sea. The groundwater beneath the proposed site is recharged by rainfall that infiltrates through the sand layer. The amount of recharge water available is dependent on a number of factors, including the rainfall quantity and intensity, the groundcover conditions, the physical catchment characteristics and the amount of evapotranspiration. Rainfall is monitored at Raglan for NIWA (Station C75731/3). The average annual rainfall at this site is 1,390 mm/year (1990 to 2000). The potential evapotranspiration (PET) is likely to be about 986 mm/year. This is based on using the PET data for New Plymouth Airport (NZ Meteorological Service, 1986), which has a similar climatic setting to Raglan. This data was used instead of data from Whatawhata, after discussions with NIWA, since the station is in a geographical setting of greater similarity to Raglan conditions than Whatawhata, despite Whatawhata being closer in distance to the site.

As there is no other water loss component (e.g. surface water), recharge to the sand layer should be about 30% of the annual rainfall (assuming recharge is 1,390 – 986 = 404 mm).

5.0 Numerical Model

The detailed assessment of effects of RI on the groundwater levels under the sand dunes has been undertaken using a groundwater numerical model. The model relies on the accuracy of field data. Considerable effort has been focussed on gathering and interpreting the field information, as discussed in Section 2.0, in order to provide a high level of confidence in the predictions from the modelling analysis.

5.1 Groundwater Modelling Program

The finite-difference groundwater flow model, MODFLOW (USGS, McDonald and Harbaugh, 1988) was used for the modelling study. This is the most widely used model internationally for groundwater modelling and has a high level of verification.

5.1.1 Modelling Approach

The following is a summary description of the approach used for developing the groundwater model. A detailed discussion of the model development is given in Appendix F, which covers the following topics.

- Developing the conceptual model
- Setting up the model mesh
- Defining the model boundaries
- Calibrating the steady state model
- Conducting predictive simulations
- Carrying out sensitivity analyses

The result (predictive simulation) is discussed in the following section.

5.1.2 Predictive Simulation

The disposal of the treated wastewater (effluent) results in the groundwater table rising or mounding. The calibrated steady state model discussed above (and Appendix F) was used to predict the mounding of the water level in the beach zone. The beach zone is the area downgradient of MW1. The maximum disposal rate that causes the water level to rise to a level of 0.5 m below the ground surface at MW1 was identified as the design-loading rate for the RI system. This was based on using mean sea level conditions and average groundwater level conditions over the period of logging to define the model parameters. The 0.5 m buffer zone between the water table and ground surface at MW1 was selected so as to provide a safety margin against seepage of water onto the surface in this vicinity. It is expected that there would be seepage of water onto the beach surface downgradient of MW1 in the vicinity of the high tide water level which is considered to be acceptable in the beach zone. The modelling found that the maximum disposal rate is about 11 m^3/d/100 m of coastline. The predicted groundwater level for such a disposal scenario is shown in Figure 2.

The modelling has assumed that the rock is isolated from the sand in terms of hydraulic connection and was not used in the model. This assumption is based on the layer of silty material (low permeability zone) between the sand and rock found in borehole MW1. If it could be shown (following further field investigation and drilling) that this material was not continuous across the rock surface, and that the rock is highly fractured with high hydraulic conductivity, then it would be valid to modify the conceptual model and incorporate the volcanic rock aquifer into the model. Using the above scenario the model would need to be recalibrated. For the recalibrated model to be valid a recharge rate higher than 30% would be required. However, a recharge rate much above 30% of rainfall does not correlate with the observed conditions at Raglan after rainfall, evapotranspiration and groundwater conditions have been taken into account. Therefore, it is considered that this scenario is unlikely.

In conclusion, because of the close correlation between the model calibration and the observed site conditions, it is considered that the present conceptual model developed in this study is reasonable and the sand layer forms the main water bearing zone underlying the site.

6.0 Conclusions

The results of the numerical modelling undertaken using the field data collected as part of this investigation show that RI is a technically feasible disposal mechanism in the site investigated, however, the allowable loading rate is very low. The numerical modelling indicates that the allowable loading is in the order of 11 m^3/d /100 m of coastline, hence for the approximately 200 m width available in the surf club area only 22 m^3/d would be able to be disposed if groundwater table rise was to be limited to an acceptable level. Although the near-surface sand saturated infiltration rates show good potential for RI, and the hydraulic conductivity measured in the monitoring well tests are of reasonable magnitude, the greatest restriction to the site in terms of potential RI use is the relatively shallow aquifer thickness, i.e. the depth of sand over the underlying rocks.

The single borehole drilled 200 m northeast of the beach access boardwalk onto Ngarunui Beach identified a similar depth of sand to that found in MW1. Therefore, it can be concluded that a similar hydraulic loading rate in the order of 11 m^3/d/100 m of coastline for an RI system in this location would likely apply.

The PDP June 2001 report estimated average future sewage flows for year 2021 to be in the order of 1,173 m^3/d. Therefore, based on an average RI disposal rate of 11 m^3/d/100 m, a coastline length in the order of 10 km would be required. Such a length of suitable coastline is not available on Ngarunui Beach, and in any case, it would be unlikely to be economically viable to construct an RI system over such a length.

7.0 References

Glover, R.E. 1964, "The Pattern of Fresh-water Flow in a Coastal Aquifer", in *Sea Water in Coastal Aquifers*, USGS Water Supply Paper 1613, pp 32-35.

Kear, D. 1960, Sheet 4 Hamilton, *Geological Map of New Zealand*, 1:250,000, DSIR, Wellington, NZ.

New Zealand Meteorological Service, 1986, *Summaries of Water Balances for New Zealand Stations,* New Zealand Meteorological Service, Wellington.

Pattle Delamore Partners Limited, June 2001, *Raglan Land Treatment Options Report*, prepared for Waikato District Council.

Appendix A

Borehole Logs

LOG OF BOREHOLE

HOLE NO. MW1

LOCATION: Ngarunui Beach, Raglan.

JOB NO: AJ81502

DATE: 22/12/2001

RL GROUND: 3.81m

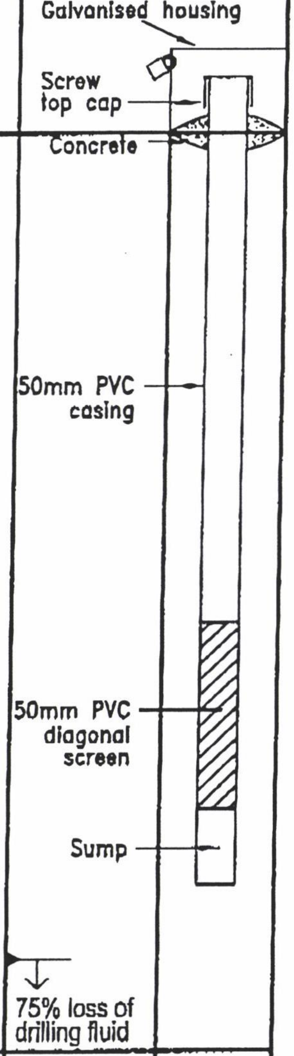
RL TOP OF CASING (PVC): 4.33m

LOGGED BY: T.S.

METHOD: Wire Line Coring

INTERPRETATION	DESCRIPTION OF SOIL/ROCK	DEPTH (m)	CORE RECOVERY %
BEACH SAND	SAND. Dark grey, well graded, loosely packed, moist, faintly bedded with occasional magnetite rich layers, fine to medium.	1.0 2.0 3.0	60 50 70 75 80 60
ALEXANDRA VOLCANICS	SANDY SILT with some gravel. Light yellowish brown, soft, moist, non-plastic, homogeneous. Gravel fine with subrounded mudstone and angular olivine andesite. (TUFF).	4.0	100
	OLIVINE ANDESITE. Grey, strong, fractured in first 0.4m, Fe stained joints.		100
	Base of borehole at 4.9m	5.0 6.0 7.0 8.0	

INSTALLATION



REMARKS: Drilled by Drillwell Exploration Ltd.

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LOG OF BOREHOLE

HOLE NO. MW2

LOCATION: Ngarunui Beach, Raglan.

JOB NO: AJ81502

DATE: 22/12/2001

RL GROUND: 6.93m

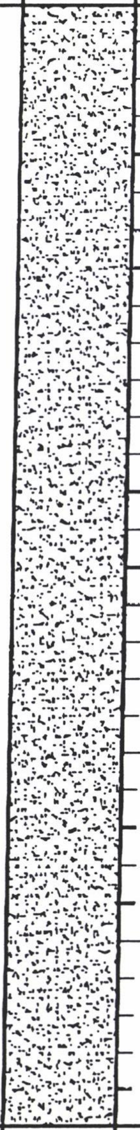
RL TOP OF CASING (PVC): 7.29m

LOGGED BY: T.S.

METHOD: Wire Line Coring

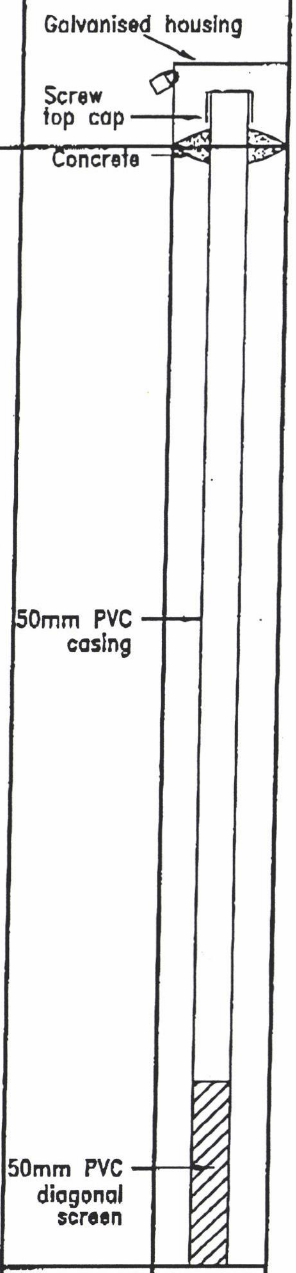
INTERPRETATION	DESCRIPTION OF SOIL/ROCK	DEPTH (m)	CORE RECOVERY %
BEACH SAND	SAND. Dark grey, well graded, loosely packed, moist, faintly bedded with occasional magnetite rich layers, fine to medium.	1.0 2.0 3.0 4.0 5.0 6.0	60 70 75 80 100 100 100 100
	Base of borehole at 6.0m	7.0 8.0	

GRAPHIC LOG



WATER LEVEL

INSTALLATION



REMARKS: Drilled by Drillwell Exploration Ltd.

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LOG OF BOREHOLE

HOLE NO. MW3

LOCATION: Ngarunui Beach, Raglan.

JOB NO: AJ81502

DATE: 23/12/2001

RL GROUND: 6.90m

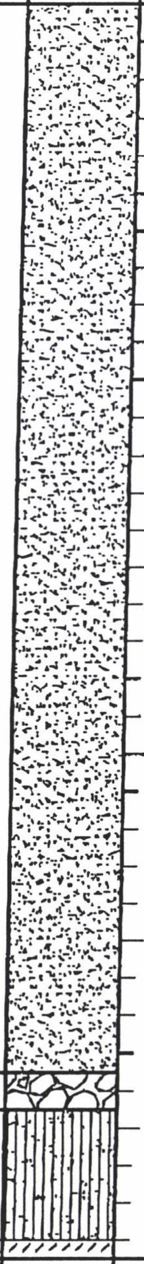
RL TOP OF CASING (PVC): 7.47m

LOGGED BY: T.S.

METHOD: Wire Line Coring

INTERPRETATION	DESCRIPTION OF SOIL/ROCK
BEACH SAND	SAND. Dark grey, well graded, loosely packed, moist, faintly bedded with occasional magnetite rich layers, fine to medium.
ALEXANDRA VOLCANICS	OLIVINE ANDESITE. Grey, strong, fractured at bottom, Fe stained joints. (VERY COARSE GRAVEL)
	SANDY SILT with some gravel. Light yellowish brown, soft, moist, non-plastic, homogeneous. Gravel fine with subrounded mudstone and angular olivine andesite. (TUFF).
	OLIVINE ANDESITE. Grey, strong, heavily fractured, Fe stained joints.
	Base of borehole at 6.7m

GRAPHIC LOG

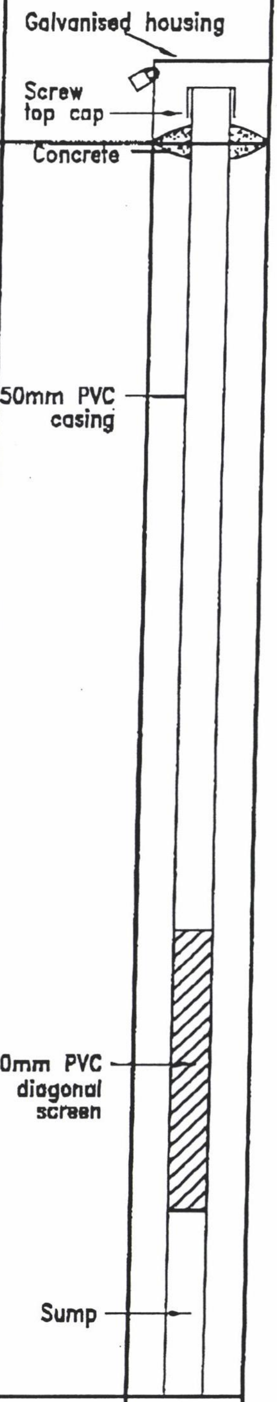


DEPTH (m): 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0

CORE RECOVERY %: 75, 70, 90, 85, 60, 85, 65, 100, 100

WATER LEVEL

INSTALLATION



REMARKS: Drilled by Drillwell Exploration Ltd.

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pdp LOG OF BOREHOLE Page 1 of 2 HOLE NO. **MW4**

LOCATION: Ngarunui Beach, Raglan.

JOB NO: AJ81502

DATE: 23/12/2001

RL GROUND: 9.48m

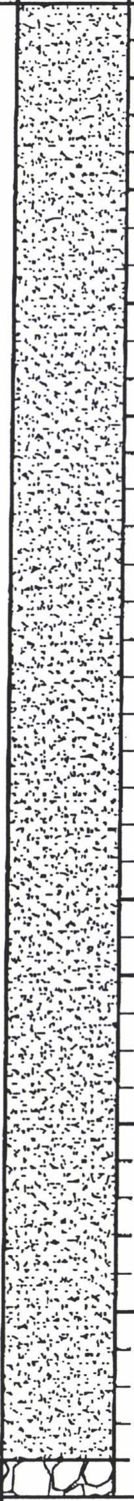
RL TOP OF CASING (PVC): 10.02m

LOGGED BY: T.S.

METHOD: Wire Line Coring

INTERPRETATION	DESCRIPTION OF SOIL/ROCK
BEACH SAND	SAND. Dark grey, well graded, loosely packed, moist, faintly bedded with occasional magnetite rich layers, fine to medium.
ALEXANDRA VOLCANICS	OLIVINE ANDESITE some silty sand. Light yellowish brown, andesite strong and heavily fractured,sand loosely packed, non-plastic, Fe stained joints. (VERY COARSE GRAVEL)

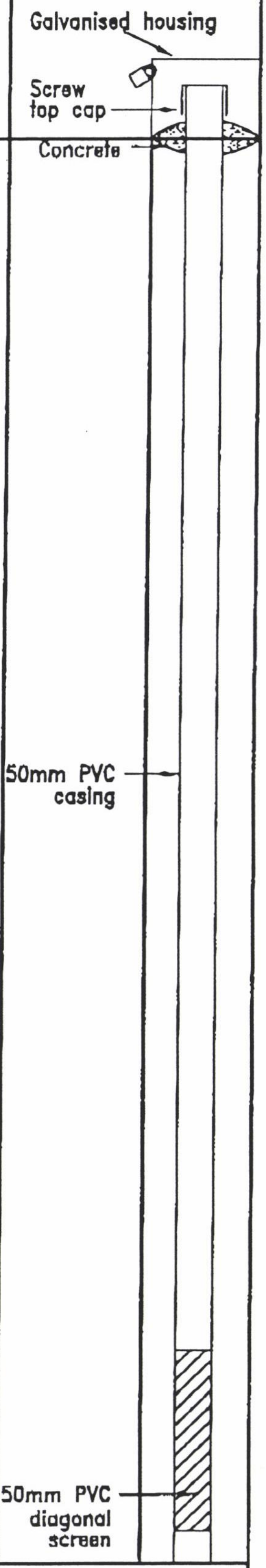
GRAPHIC LOG



DEPTH (m)	CORE RECOVERY %
	45
1.0	
	50
2.0	
	85
3.0	90
4.0	65
5.0	75
	60
6.0	70
7.0	75
	70
8.0	100

WATER LEVEL

INSTALLATION



REMARKS: Drilled by Drillwell Exploration Ltd.

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LOG OF BOREHOLE

Page 2 of 2

HOLE NO. MW4

LOCATION: Ngarunui Beach, Raglan.

JOB NO: AJ81502 DATE: 23/12/2001

RL GROUND: 9.48m

RL TOP OF CASING (PVC): 10.02m

LOGGED BY: T.S. METHOD: Wire Line Coring

INTERPRETATION	DESCRIPTION OF SOIL/ROCK	GRAPHIC LOG	DEPTH (m)	CORE RECOVERY %	WATER LEVEL	INSTALLATION
ALEXANDRA VOLCANICS	SANDY SILT with some gravel. Light yellowish brown, soft, moist, non-plastic, homogeneous. Gravel fine with subrounded mudstone and angular olivine andesite. (TUFF)	▥		100		50mm PVC casing
	OLIVINE ANDESITE. Grey, strong, homogeneous.	▨				Sump
	Base of borehole at 8.3m		9.0			
			10.0			
			11.0			
			12.0			
			13.0			
			14.0			
			15.0			
			16.0			

REMARKS: Drilled by Drillwell Exploration Ltd.

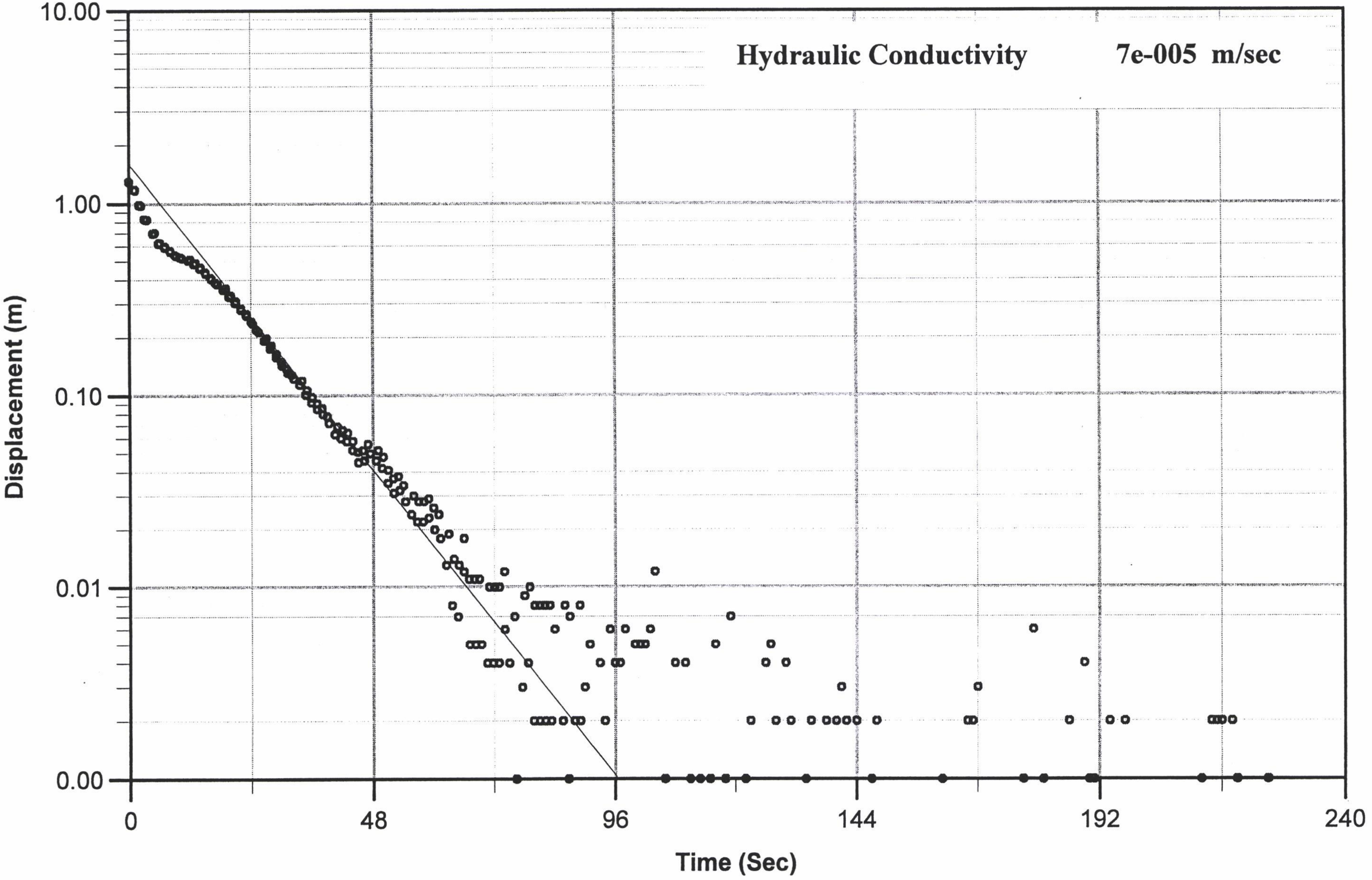
PATTLE DELAMORE PARTNERS LTD

F:\DATA\JOBS\AJ815\02\Drawings\MW 1-4.dwg

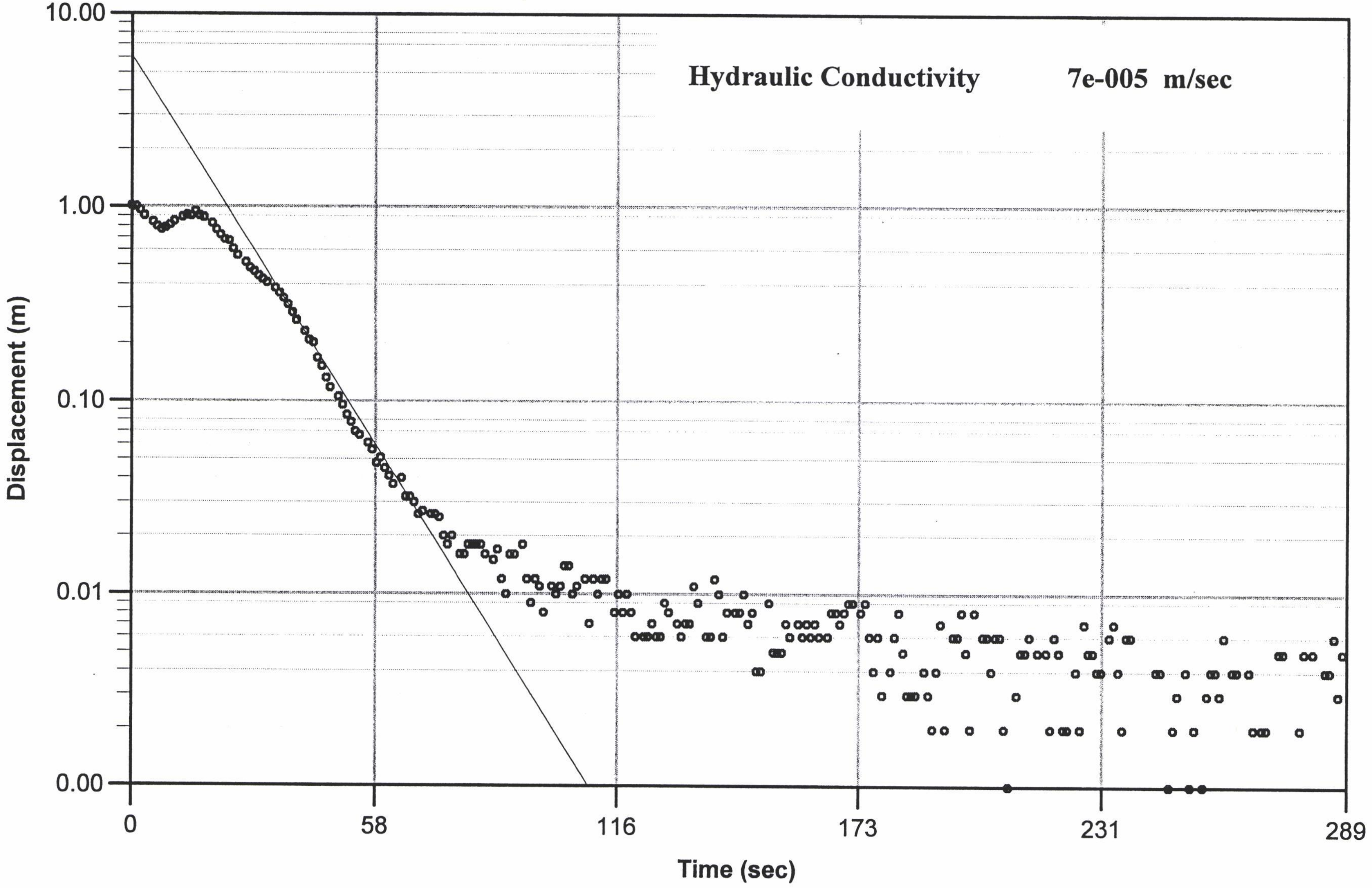
Appendix B

Slug Test Analysis Results

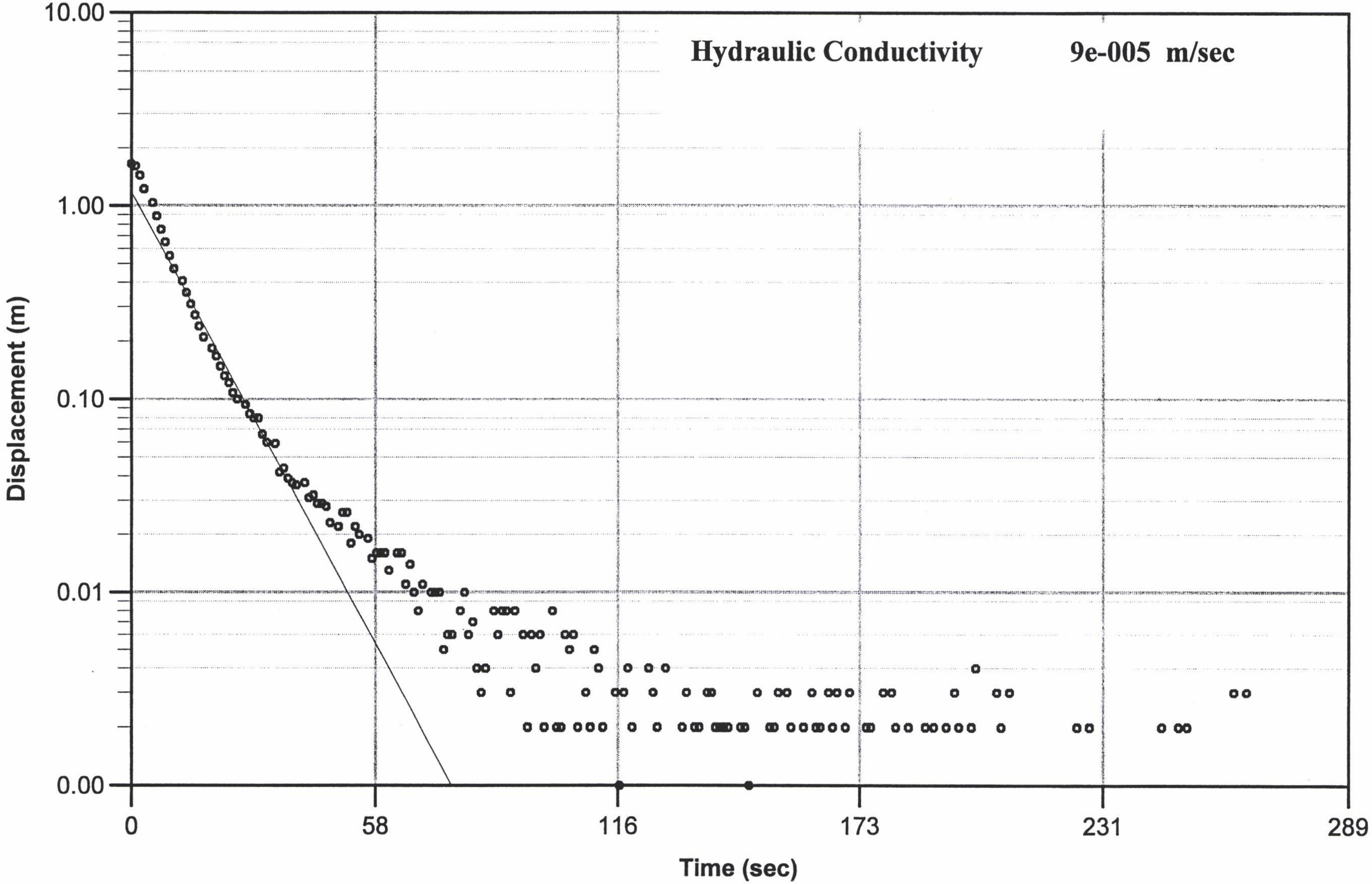
MW1 First Test (Bouwer and Rice)

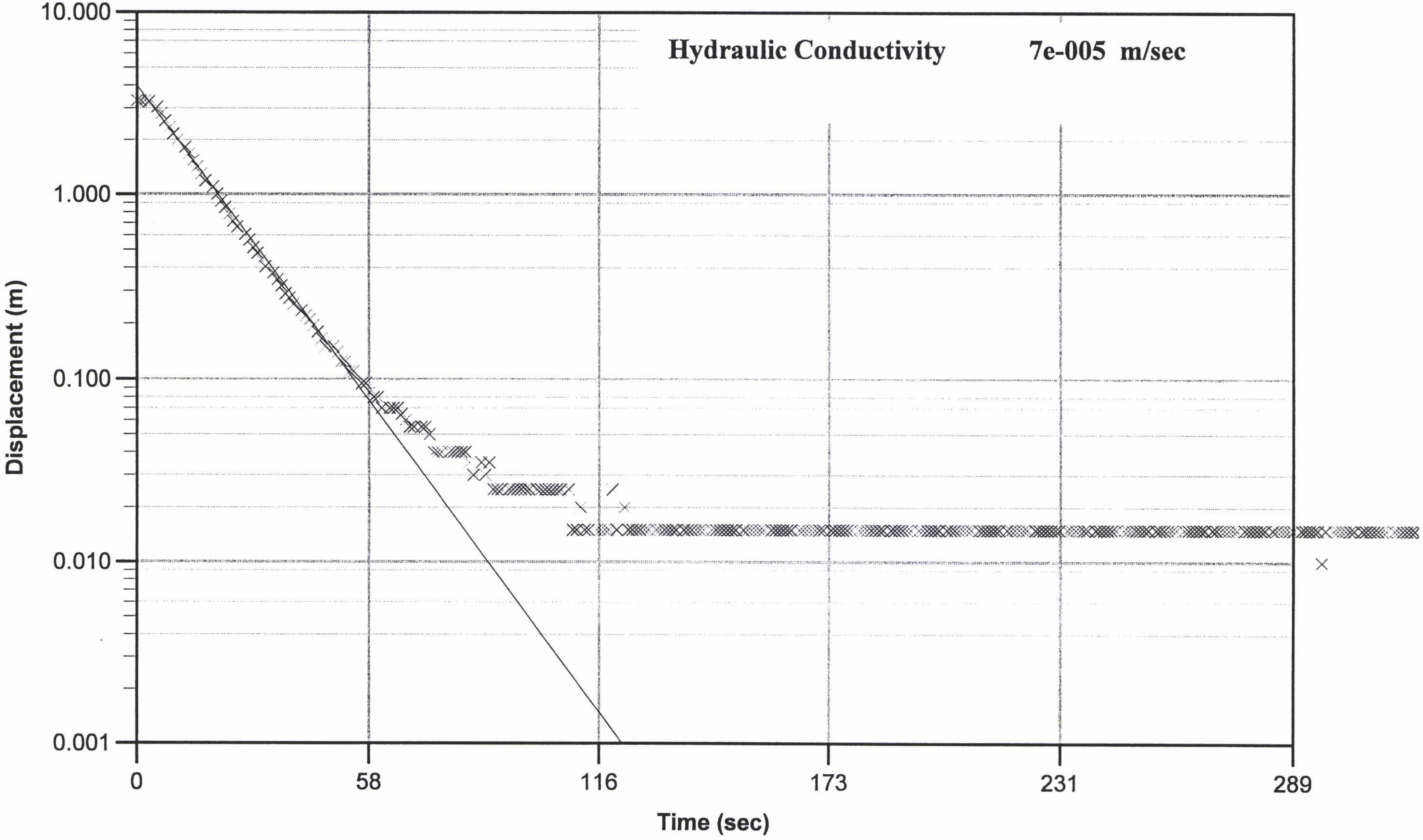


MW1 Second Test (Bouwer and Rice)



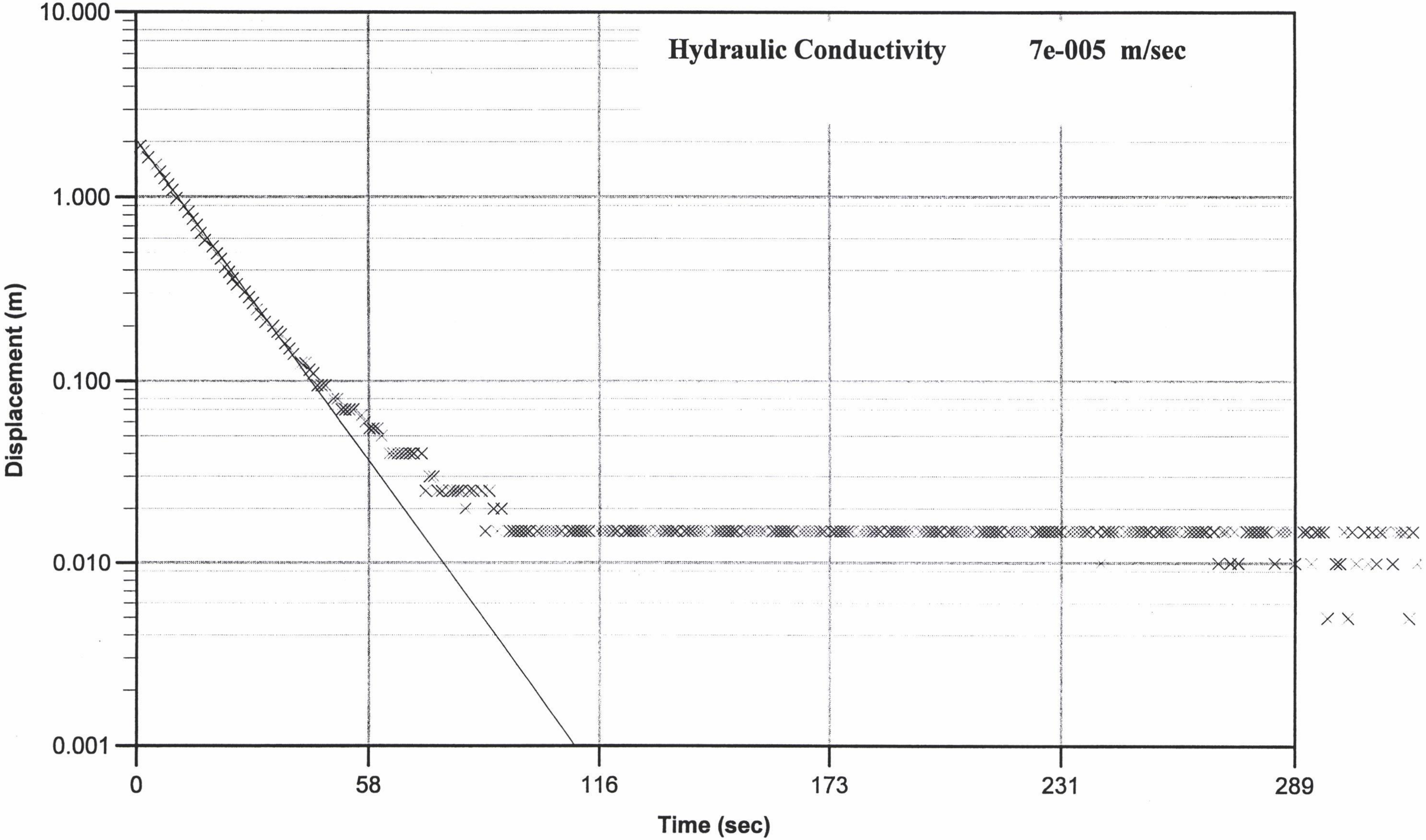
MW1 Third Test (Bouwer and Rice)



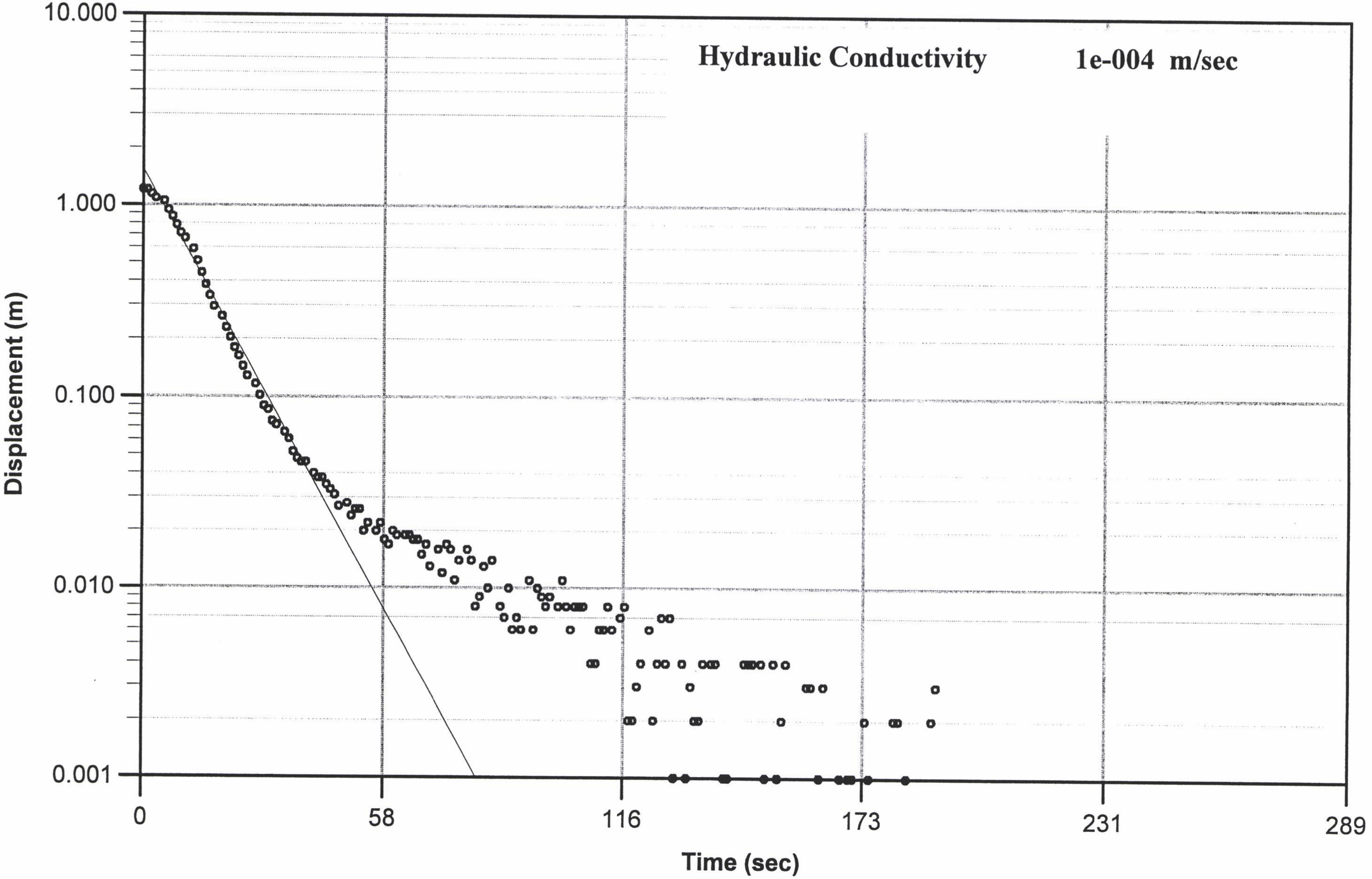


MW2 First Test (Bouwer and Rice)

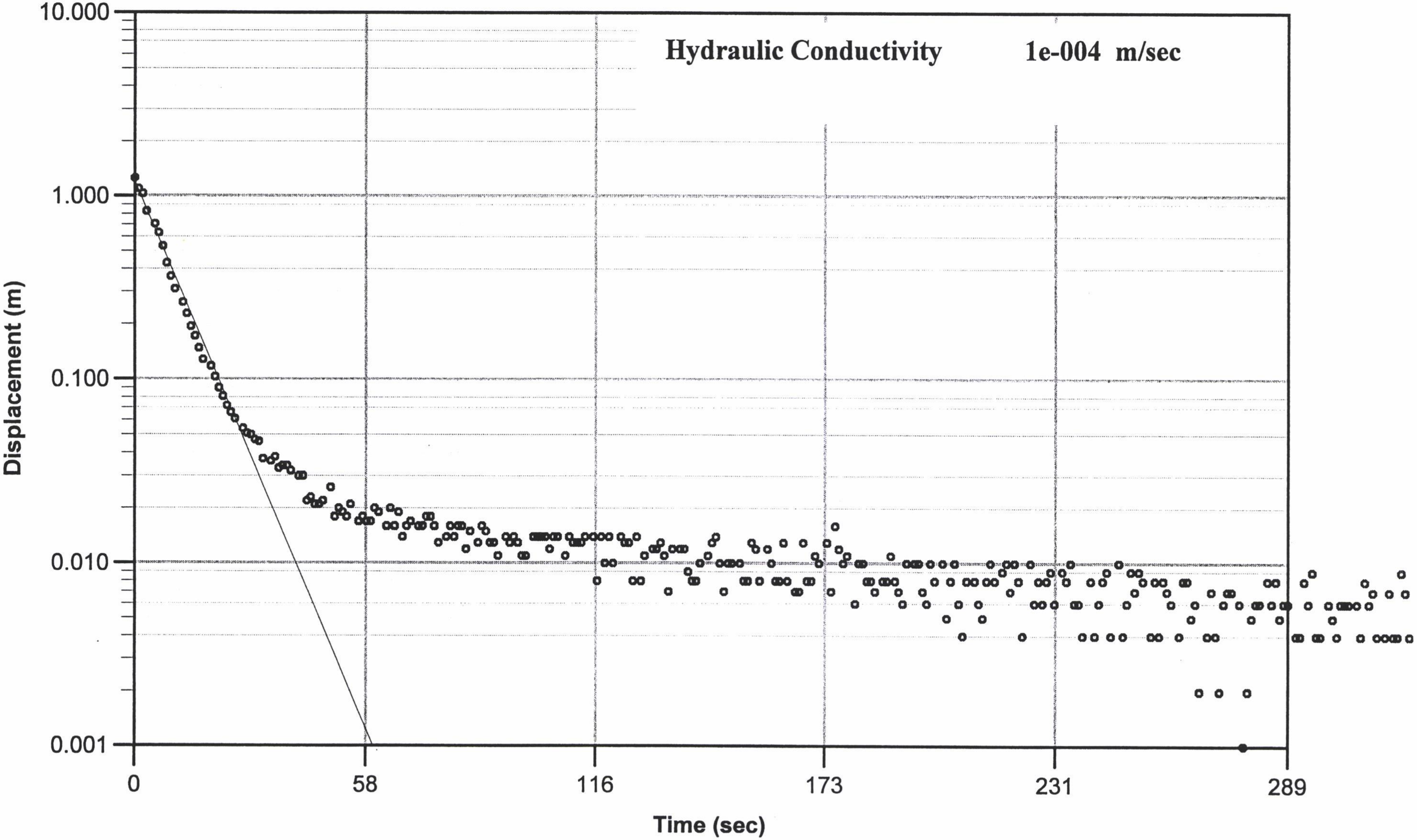
MW2 Second Test (Bouwer and Rice)



MW3 First Test (Bouwer and Rice)

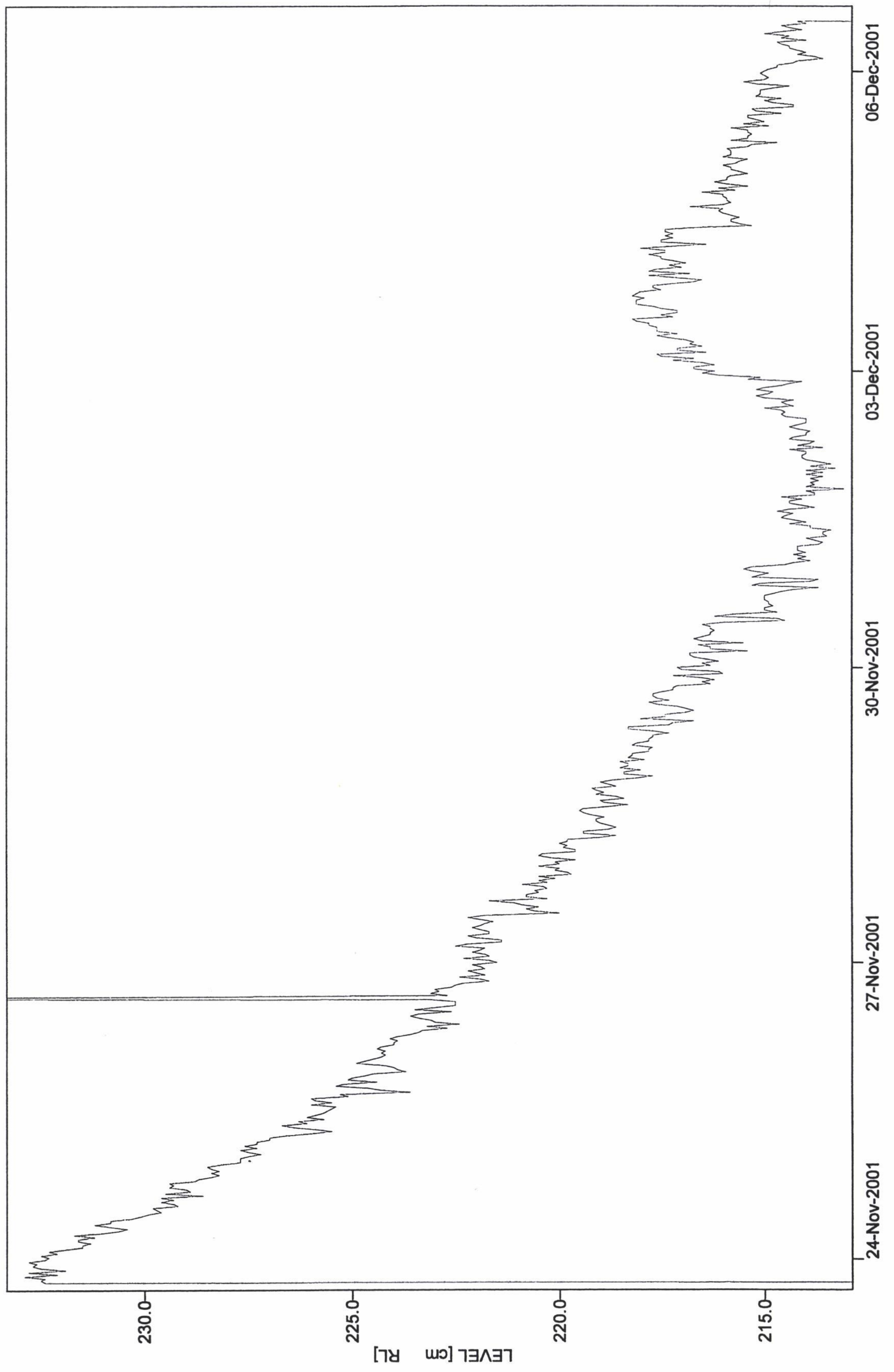


MW3 Second Test (Bouwer and Rice)



Appendix C

Water Level Monitoring Results and Rainfall



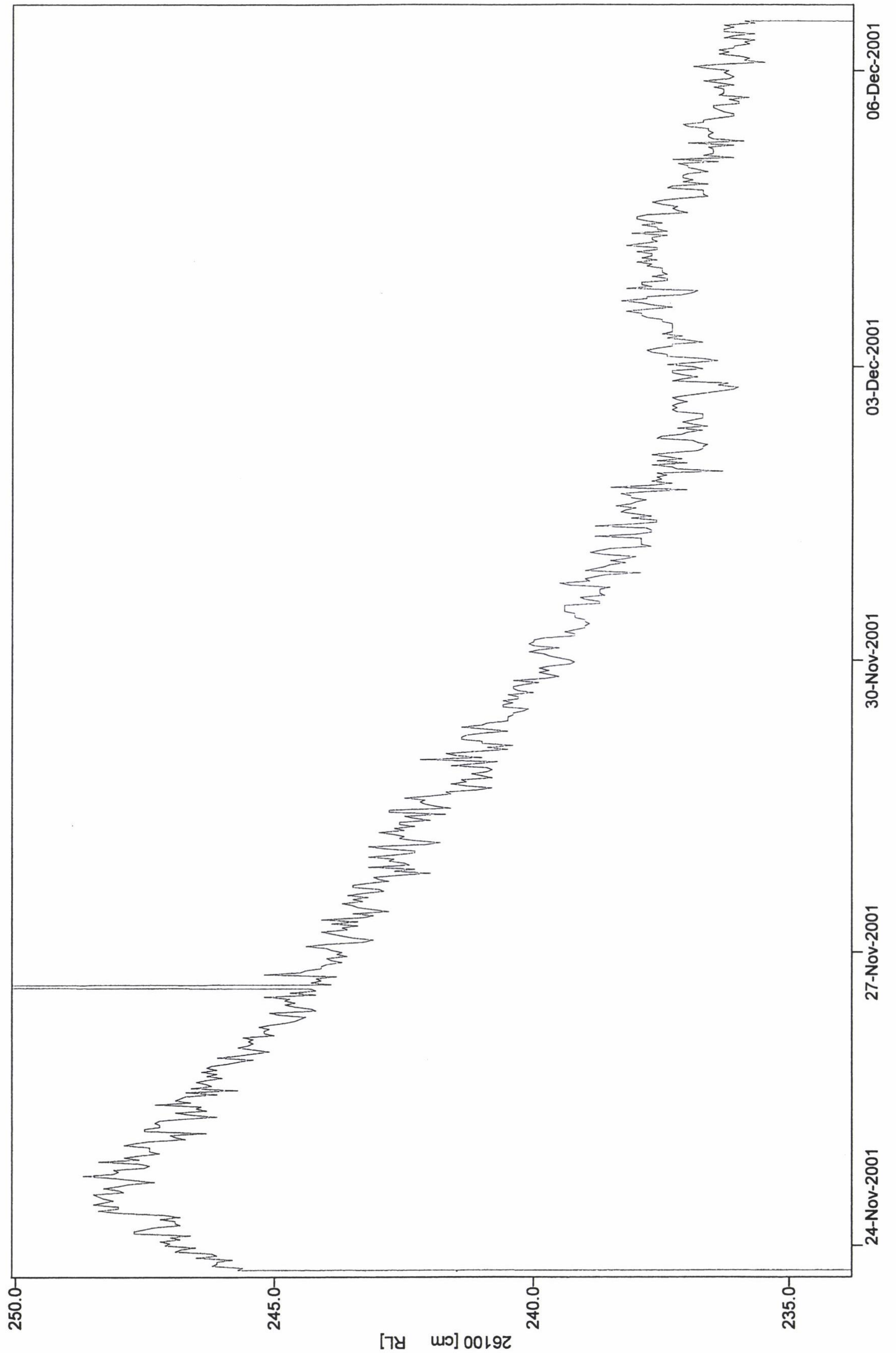
Pattle Delamore
PDP

date

2002-01-14

L-Levelogger (06747), Instrument number: 6747
Location: raglan MW1

A4

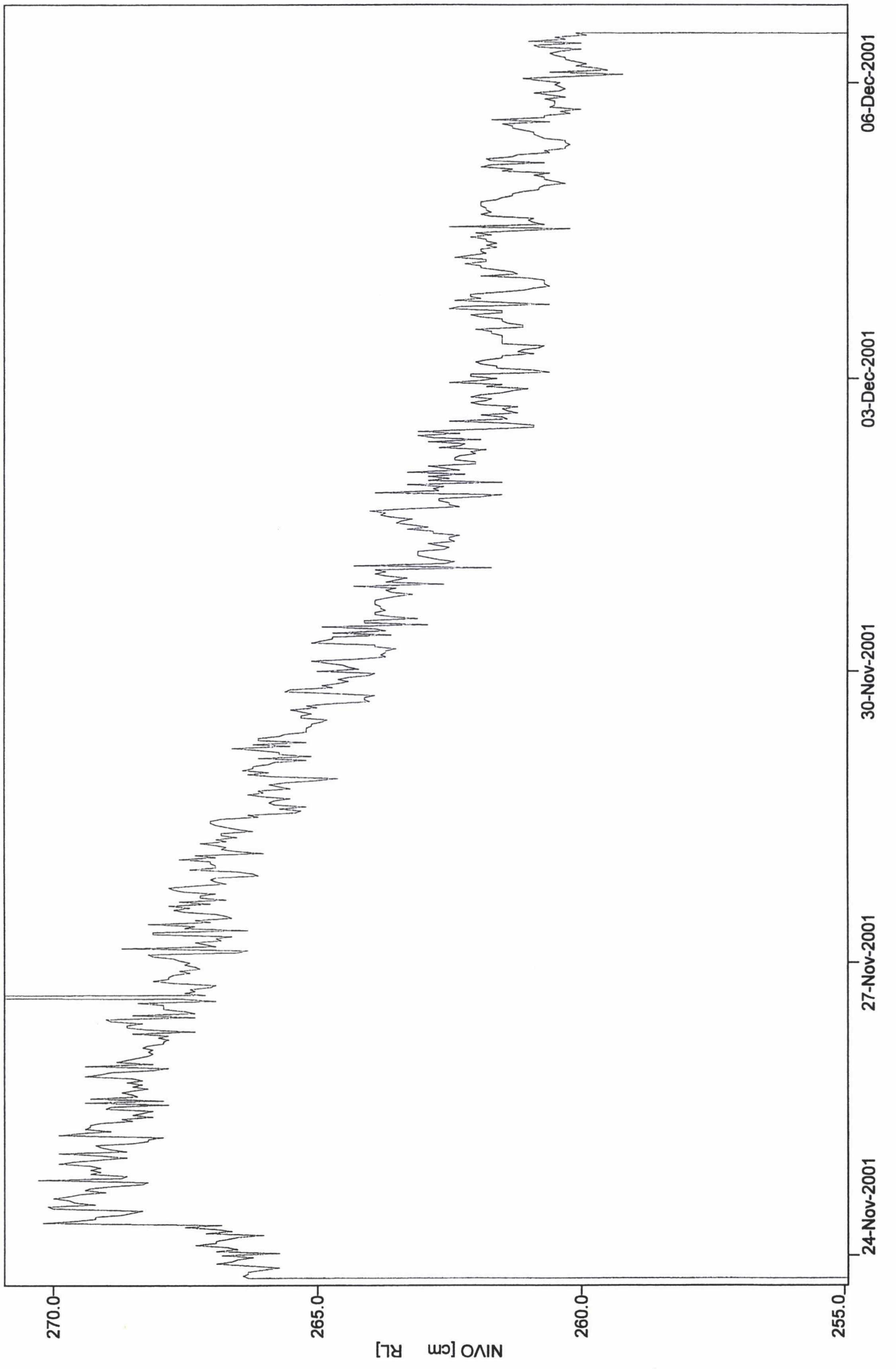


Pattle Delamore
PDP

date
2002-01-15

L-Levelogger (05661), Instrument number: 5661
Location: raglan MW2

A4



Pattle Delamore
PDP

date
2002-01-15

L-Levelogger (06204), Instrument number: 6204
Location: raglan MW3

A4

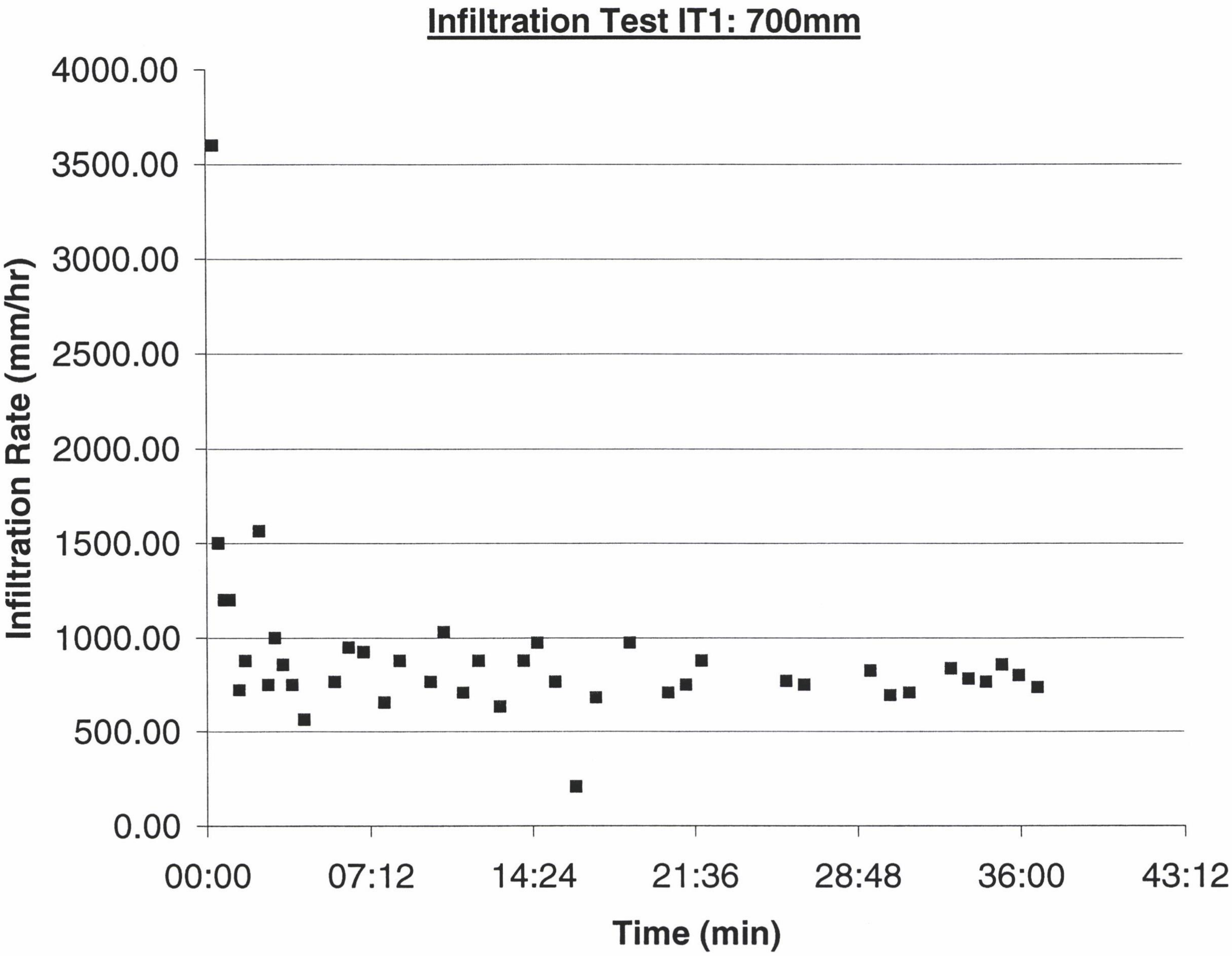
Table C1: Depth to Groundwater	
Well	**Depth to Water [1.] (m)**
MW1	1.67
MW2	4.57
MW3	4.30
MW4	6.72

1. (Depth below ground level)

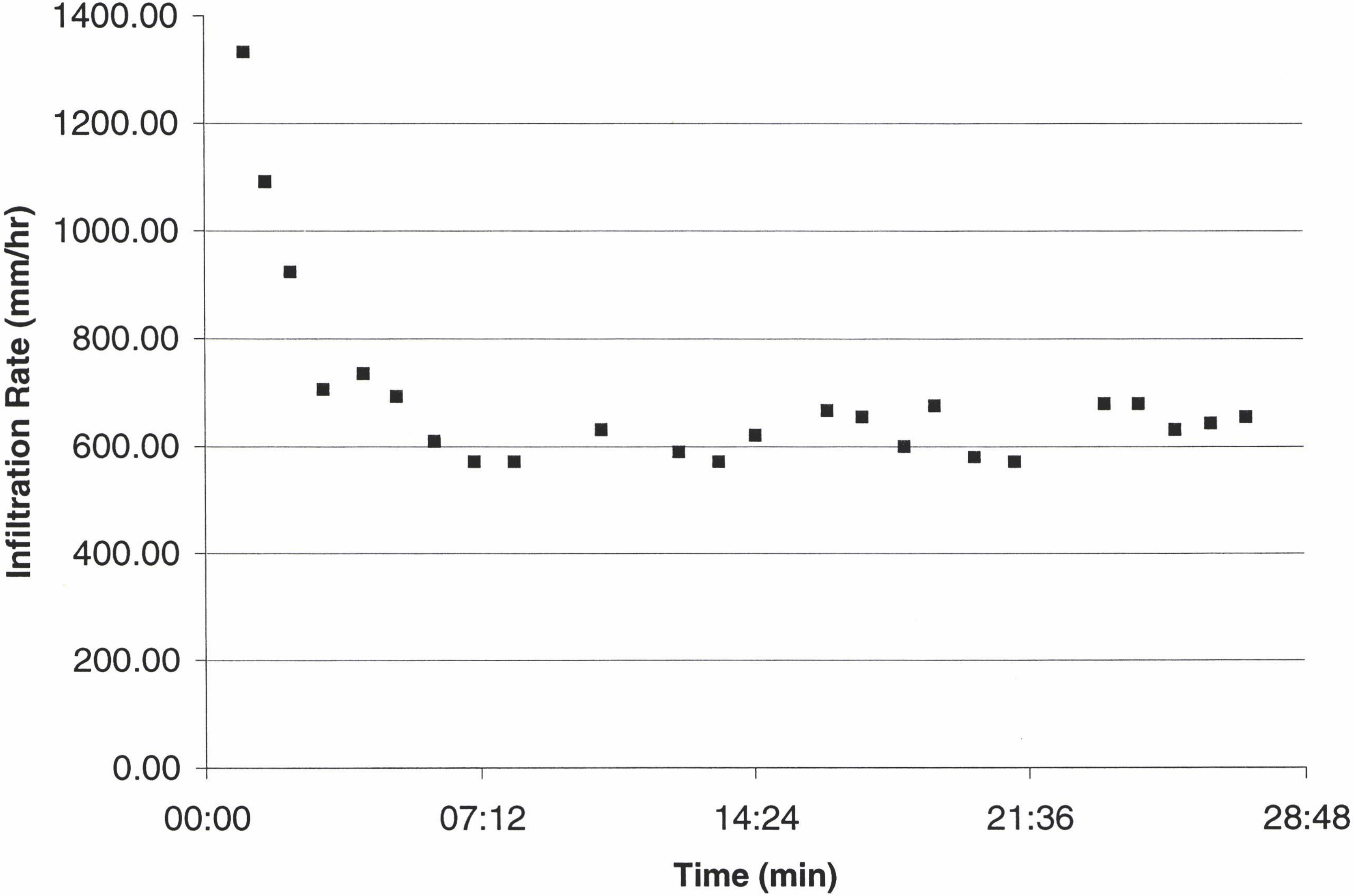
Table C2: Rainfall at Raglan	
Date (2001)	**Rainfall (mm)**
November	
20	0
21	12.7
22	27
23	0
14	0
25	0
26	0
27	2
28	0
29	0
30	2.2
December	
1	3.6
2	11
3	0
4	0
5	0.2
6	7.7
7	48.9
8	5.3

Appendix D

Infiltration Test Results


Infiltration Test IT1: 700mm
Infiltration Rate (mm/hr)
4000.00
3500.00
3000.00
2500.00
2000.00
1500.00
1000.00
500.00
0.00
00:00
07:12
14:24
21:36
28:48
36:00
43:12
Time (min)

Infiltration Test IT2: 300mm



Appendix E

Grading Curve Results

ENVIROLAB
GEOTEST

Pattle Delamore Partners Ltd

PO BOX 9528
NEWMARKET

12 December 2001

Our Ref: 6006108

L1:53898-JDG17L01.DOC

RECEIVED
1 [illegible] 2001

Attention: Andrew Sussex

Dear Sir

Dry Grading – Raglan AJ815

Please find the final results for the testing carried out on sample IT1 (report 1126L:03). Any queries please don't hesitate to contact me on the number below.

Yours faithfully
Envirolab Geotest Ltd

Justin Gisel

Direct Dial: +64-9-300 9005
Email: jgisel@beca.co.nz
JDG:jdg

ENVIROLAB GEOTEST LIMITED
Environmental and Geotechnical Laboratory
Water, Wastewater and Soils Investigations, Air Pollution Monitoring, Earthworks Control

117 Vincent Street, PO Box 5585, Auckland, New Zealand.
Tel +64-9-300 9200 Fax +64-9-300 9292

AS/NZS ISO9001 BVQi ISO GUIDE 25

SUMMARY OF SOIL TEST RESULTS

Report: 1126L:03

Sheet 1 of 2

ENVIROLAB GEOTEST

132 Vincent Street
P O Box 5585
Auckland
Ph. 300-9005
Fax. 300-9300

Job Name: Raglan AJ815

Job No: 6006108/089

Client: Pattle Delamore Partners Ltd

Date: 11 December 2001

Test Pit No.	Sample No.	Depth (m)	Sample Type	Sample Description	Natural		Atterberg Limits		Grading	Ps t/m^3	Clay Index	Consol	CBR's	Compaction	Perm k m/s	Triaxial CUPP
					Mc%	Bulk Density t/m^3	LL/ CPL	PL								
-	IT1	0.7-0.8	BULK	Dark brownish grey fine to medium SAND; moist, non-plastic.					X							

new zealand accredited laboratory

ENVIROLAB GEOTEST IS REGISTERED BY INTERNATIONAL ACCREDITATION NEW ZEALAND. ALL TESTS REPORTED HEREIN HAVE BEEN PERFORMED IN ACCORDANCE WITH THE LABORATORY'S TERMS OF REGISTRATION . THIS REPORT MAY NOT BE REPRODUCED EXCEPT IN FULL
NOTE: IANZ ENDORSEMENT DOES NOT COVER SOIL DESCRIPTIONS.

REPORT RELATES ONLY TO SAMPLES TESTED, SAMPLING WAS UNDERTAKEN BY OTHERS.

X DATA ATTACHED, BULK = BULK SAMPLES

TEST STANDARDS:

NZS 4402: 1986; Test 2.8.2

AUTHORISED SIGNATORY

A. Pickard – Geotest Lab Supervisor

Sheet 1 of 2

ENVIROLAB
GEOTEST

132 Vincent St
PO Box 5585
Auckland
Ph (09) 300 9005
Fax (09) 300 9300

PARTICLE SIZE DISTRIBUTION

Job Name: Raglan AJ815

Client: Pattle Delamore Partners Ltd

Date: 11 December 2001

Job No.: 6006108/089

Tested By: J. Gisel

Checked By: B. Lobb

Test Pit No.: -

Sample No.: IT1

Depth (m): 0.7-0.8

Sample Type: Bulk

History: As Received

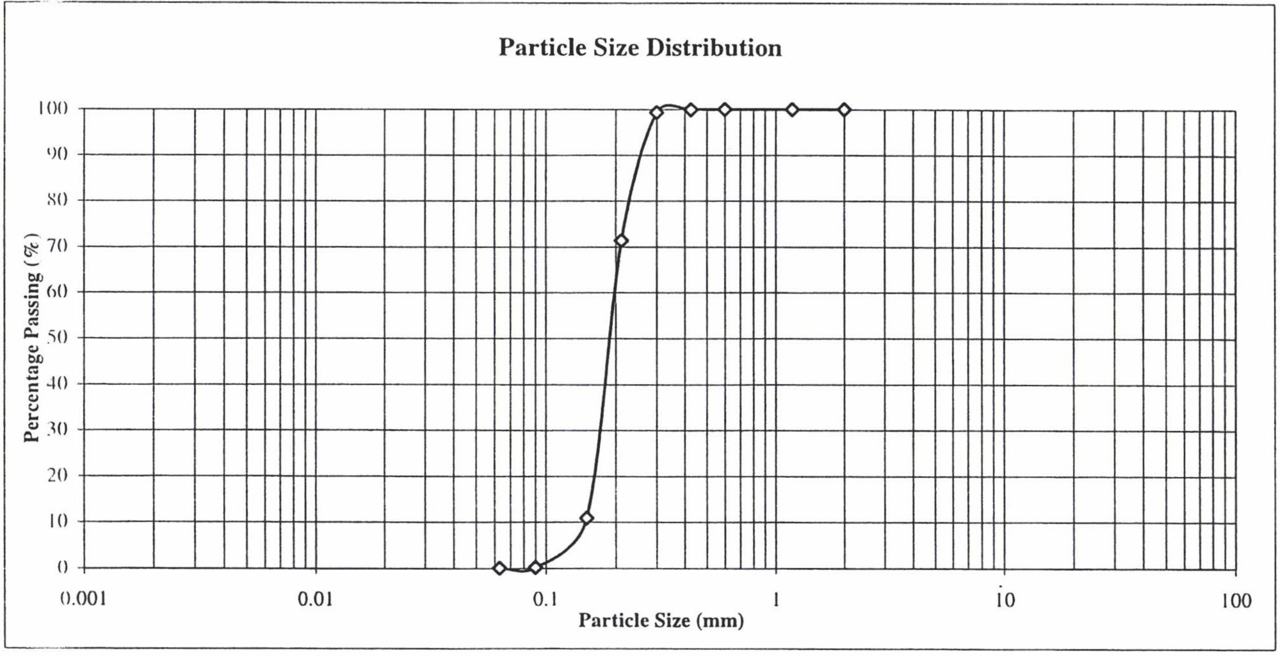
Report No: 1126L:03

Sample Description: Dark brownish grey fine to medium SAND; moist, non-plastic.

Test Standard: NZS 4402:1986, Test 2.8.2

Coarse & Intermediate Fraction		Fine Fraction	
Sieve Size	% Passing	Sieve Size	% Passing
75mm		2mm	100
63mm		1.18mm	100
53mm		600µm	100
37.5mm		425µm	100
26.5mm		300µm	99
19mm		212µm	71
13.2mm		150µm	11
9.5mm		90µm	0
6.7mm		63µm	0
4.75mm		<63µm*	0

*Mass passing 0.063mm obtained by difference



Authorised Signatory:.................................

A. Pickard - Geotest Lab Supervisor
P:\62\6006108\grading\IT1.xls

Appendix F

Numerical Modelling

Appendix F

1.0 Numerical Modelling

1.1 Developing Conceptual Model

The conceptual hydrogeological model forms the basis for developing the groundwater flow system in this area. The key aspects of the conceptualisation are:

- A geological framework comprising about 4m (at the beach) of a fine sand layer underlain by the andesite/basalt rocks of the Alexandra Volcanics Formation. The geometric mean of the hydraulic conductivity of the sand is 8×10^{-5} m/s (based on the slug tests results).
- Groundwater flow is from the hill, i.e. the exposed weathered andesite/basalt rocks, towards the sea.
- Recharge to the groundwater system occurs primarily via rainfall infiltration.
- Recharge in the sand layer is expected to be approximately 30% of rainfall.
- Discharge from the groundwater flow system occurs via seepage to sea.

1.2 Setting up the model grid

The conceptual model is converted to a grid-based numerical model using data from 4 boreholes (Appendix A) and surface topography.

The model consists of one layer, i.e. the sand layer, as the main bulk of groundwater is expected to move through the sand layer. The underlying volcanic rock is assumed to be inactive i.e. not hydraulically connected.

The model consists of a total of nodes of 237 with the node spacing of 2 m.

1.3 Defining the model boundaries

The flow paths in the area are perpendicular to the coastline. Therefore a 2d model perpendicular to the coastline can represent the groundwater conditions.

The model grid covers roughly a rectangular area of about 470 m (length) by 1 m (width).

The following boundaries are considered for the groundwater model:

Constant head nodes simulating mean sea level (RL =0 m). The north and south no-flow boundaries are assigned parallel to flow lines. The east boundary of the model is assigned along the contact between the sand layer and the exposed weathered rocks.

Borehole logs have been used to assign the lower boundaries of the sand layer.

1.4 Calibrating the steady state model

During the calibration process, the most uncertain model parameters, i.e. recharge, were varied until the model produced similar answers to the measurements that have been made in the field during the site investigation. Recharge was altered within plausible ranges (20 to 40% of the annual rainfall) until the computed water levels matched the field data as closely as possible. The calibrated model uses a recharge rate of 30% of the annual rainfall. This is nearly identical to the recharge estimation made using rainfall and evapotranspiration data. The steady state model output is shown Figure F1.

The calculated versus computed heads is shown in Figure F2. The calibration residual of 0.15 m (RMS) is achieved.

An analytical method (Modified Dupuit method for an unconfined aquifer with a sloping base) was used to check the model water balance. The flow predicted by the model for the existing natural state conditions (i.e. prior to any RI being undertaken) is close to the value calculated using the analytical method (Section 4.1.1), i.e. 0.28 m^3/d/1 m of the coastline.

1.5 Conducting Predictive Simulation

The predictive simulation is used to assess the effects of the proposed RI on the water levels on the beach area (close to MW1). The results are discussed in Section 5.1.2.

1.6 Carrying out Sensitivity Analyses

Sensitivity analyses were undertaken to determine the range of uncertainty in the calibrated model. The analyses indicate that the model is highly sensitive to changes in hydraulic conductivity.

1.6.1 Hydraulic conductivity

Increasing the hydraulic conductivity of the sand layer to its maximum measured values (1 x 10^{-4} m/s) without changing any other parameters results in about 0.5 m drop in measured hydraulic heads (Figure F3).

Reducing the hydraulic conductivity of all units to minimum plausible values (5 x 10^{-5} m/s) results in about 1m increase in hydraulic heads.

The assigned hydraulic conductivity value produces the best calibration result (Figure F3).

1.6.2 Recharge

Recharge to the sand layer is not expected to be more than 40% or less than 20% of the annual rainfall. The variation in recharge within the plausible range causes changes in computed heads from –1 m (using the minimum recharge rate) to +0.5 m (using the maximum recharge rate). The results are shown in Figure F4.

The values used during the calibrations are similar to the estimated values gathered during the conceptualisation and therefore a high degree of confidence can be expected from the model predictive simulations.

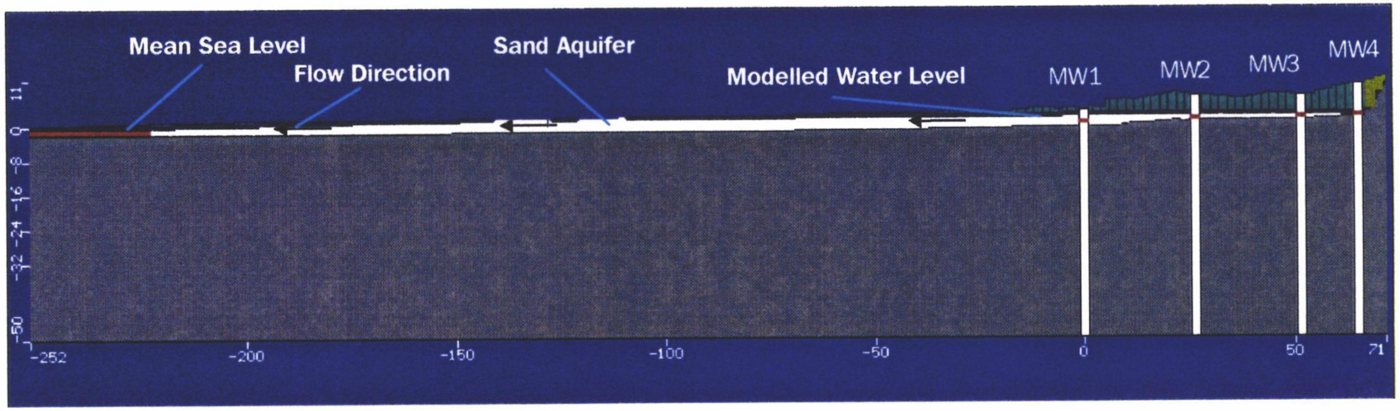


Figure F1: Steady State Model Output (water levels)

Figure F2: Calibration Results

RMS=0.15m

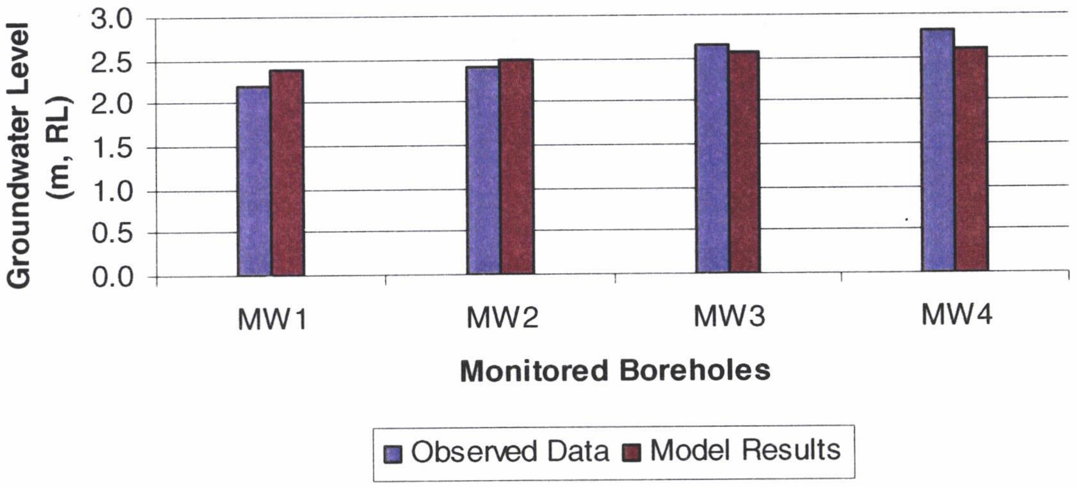


Figure F3: Sensitivity Analysis

Response of Hydraulic Head to Changes in Hydraulic Conductivity

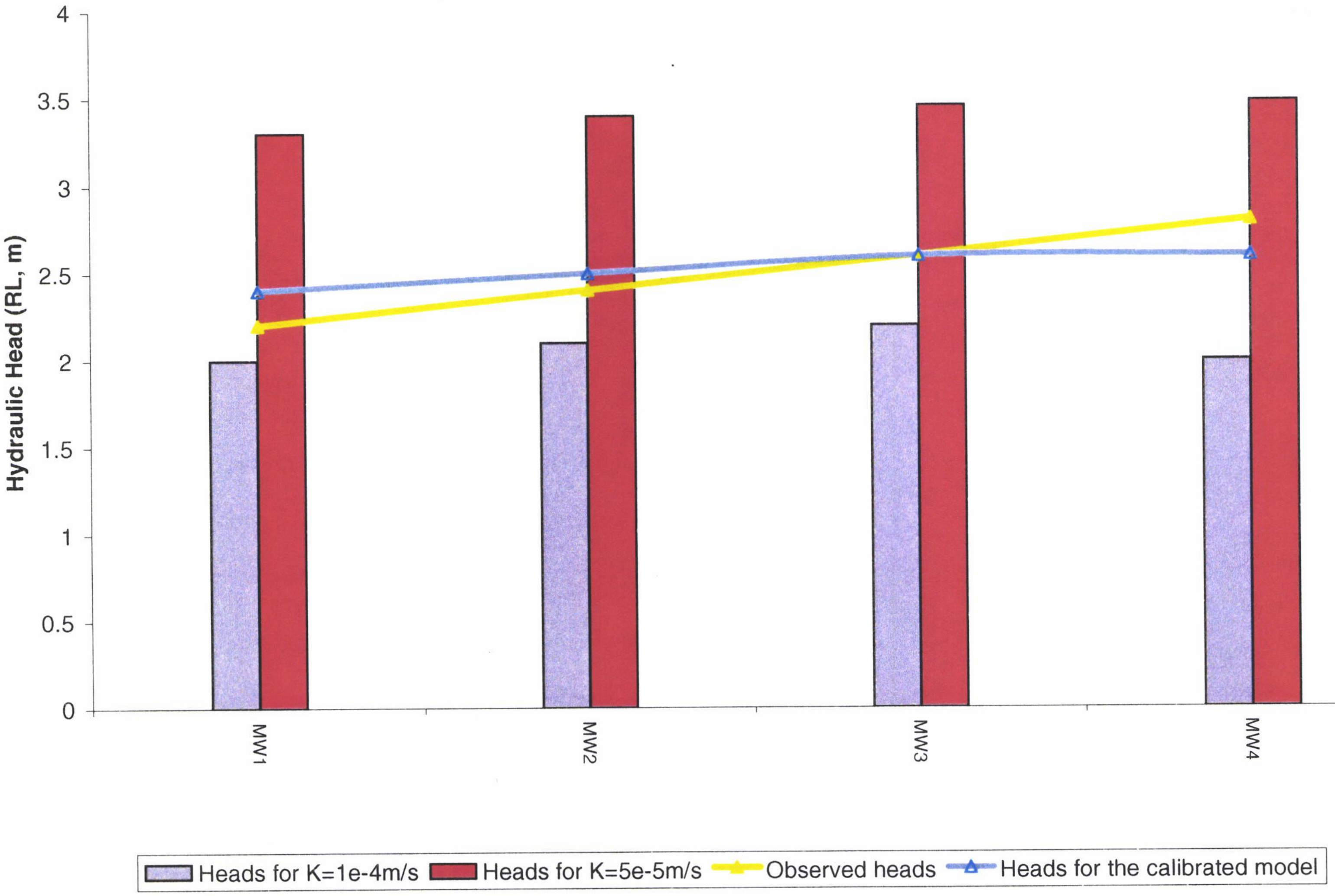


Figure F4: Sensitivity Analysis

Response of Hydraulic Head to Changes in Recharge