Yamba Hill groundwater level monitoring assessment to mid-2007

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YAMBA HILL GROUNDWATER LEVEL MONITORING ASSESSMENT TO MID-2007

by

W A Timms, I L Cunningham and J T Carley

Technical Report 2007/32
December 2007

THE UNIVERSITY OF NEW SOUTH WALES
SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING
WATER RESEARCH LABORATORY
in association with
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and
GROUNDWATER DATA COLLECTION SERVICES

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1. INTRODUCTION

The critical issue of slope stability was identified by the Yamba Coastline Management Study undertaken by Manly Hydraulics Laboratory (MHL 2002a,b, 2003). The study area (Figure 1), from the northern end of Pippi Beach to Turners Beach adjacent to the Clarence River breakwater, features several dunes and bluffs backing sandy pocket beaches.

The Water Research Laboratory (WRL) team includes geotechnical engineers from Jeffrey and Katauskas Pty Ltd (J&K) and monitoring services by Groundwater Data Collection Services (GDCS). This study is funded by Clarence Valley Council (CVC) who have received a grant from the Natural Disaster Mitigation Program 2003/04 as part of implementing the Coastal Management Strategy (CVC, 2004).

This report should be read in conjunction with the first project report (WRL Technical Report 2005/34), which provided information on monitoring installations. This report presents the following information:

- Monitoring results for the period 26th May 2005 to 25th May 2007 (project managed by WRL)
- A hydrogeological assessment and a coastal processes and hazards assessment (by WRL)
- A geotechnical stability analysis and an overall landslide risk analysis (Appendix A includes the associated report by J&K).

This report shows that groundwater levels respond to major rainfall events, with the potential to trigger landslides. No ground movement was detected except for one site where 5 mm of movement indicated a slow creep at the toe of the slope. A revised hydrogeological conceptual model, on the basis of observed conditions was very similar to a preliminary conceptual model. The revised assessment found landslide risk was unacceptably high for rapid to very rapid movements in some areas, with tolerable and acceptable risk levels in other situations. Emergency management strategies should remain in place until more permanent stabilisation measures are adopted, while groundwater level and inclinometer monitoring should continue.

Another project report will be issued in the future if CVC decide to continue monitoring.

1.1 Background

Landslide Risk Zones (LRZs) and stability strategies were defined based on existing data as part of the Yamba Coastline Management Study (MHL 2002a, 2002b and 2003). Subsequent geotechnical investigations have focused on the slopes above Yamba beach and on the Pacific Hotel grounds. Detailed site investigations and monitoring have not been carried out at northern Convent Beach (Jeffrey and Katauskas 2000). At the time of this report, subsurface drainage works and site specific groundwater monitoring had been implemented by the Pacific Hotel.

The objective of this project was to obtain site specific investigation and monitoring data and to undertake a review of the LRZs. The response of groundwater levels to major storm events was a major focus, having been identified as a likely trigger mechanism for hill slope instability (AGS, 2000).

1.2 Structure of this Report

This report should be read in conjunction with WRL Technical Report 2005/34 which presents details of piezometer and inclinometer monitoring installations, included detailed bore logs, elevation survey, and particle size analysis. Community information newsletters No. 1, 2 and 3 were provided in an Appendix to this previous report. Newsletter No. 4 is attached as Appendix D.

This report is comprised of a groundwater assessment and geotechnical assessment (Appendix A). Section 2 of this report is a summary of monitoring installations while Section 3 summarises monitoring data available. Details of groundwater response to a major rainfall event is provided in Section 4, followed by a coastal processes and hazard assessment in Section 5. All of this information is used in the hydrogeological assessment (Section 6) and geotechnical assessment (Appendix A, summarised in Section 7). The report concludes with recommendations for continued monitoring and slope stabilisation (Section 8).

2. MONITORING SITE INSTALLATIONS

The Yamba Hill site is located in northern New South Wales (Figure 1), above Main Beach and the Yamba Surf Life Saving Club. Yamba Hill is a coastal dune approximately 30 m high. The dune overlies weathered sandstone and a cliff approximately 6-8 m high above Main Beach (Figure 3). The slopes above the cliff line toe are between 18-35°, and vegetated by scrub/bushes (Jeffrey and Katauskas, 2000). Residential lots and the Pacific Hotel are located above a flatter bench area which rises from Marine Parade behind the Surf Life Saving Club. This Yamba hillslope was identified as the highest LRZ in the Yamba Area (Jeffrey and Katauskas 2000, MHL 2002a, 2002b and 2003).

Piezometers and inclinometers were installed to monitor groundwater levels and hill slope stability. A summary of the installations is shown in Table 1 and the locations are shown in Figure 2. Three monitoring transects were orientated perpendicular to the beach as follows:

- Transect 1 (southern) was located on a reserve south of the Pacific Hotel, with installations at the top, mid and base of slope
- Transect 2 (middle) was located on Crown Land, with installations at the top and base of the slope
- Transect 3 (northern) was located on Crown Land including an access pathway, with installations at the top, mid and base of slope.

The piezometers and inclinometers installed along each transect are detailed in Section 2.1. Groundwater level loggers were also installed in three existing monitoring bores on the Pacific Hotel Property (MSA1, MSA5 and MSA9). The approximate locations of these bores are indicated on Figure 2.

2.1 Piezometers and Inclinometers

A network of 8 piezometers and 6 inclinometers were installed in April 2005 with assistance from North Coast Drilling (formerly Craig Pullman Site Investigations). A 100 mm diameter auger system was used in washbore style, with sampling of sediments by SPT (standard penetrometer testing) and coring of sandstone.

Piezometers were constructed using 50 mm Class 18 PVC, with a 0.75 m screened interval at the base of the hole. Holes were allowed to collapse naturally and no backfill or gravel-pack was used. Where washbore methods were used, holes were completed by backflushing with a mild bleach to aid in breakdown of the drilling mud to allow natural collapse.

Inclinometer casings were installed using 70 mm casing and grouted from the base to the ground surface using a tremmie pipe system. The base of the inclinometer casings were anchored in cored holes within sandstone.

Table 1
Summary of Installations

Type	**Site**	**Location**		**RL Surface**	**RL Base of Piezo***	**Depth of Piezo#**	**Drill Method**
		mE	***mN***	***mAHD***	***mAHD***	***mBGL***	
Piezometers	1A	535239.34	6743756.73	29.47	17.90	11.58	Washbore
	1B	535257.60	6743754.70	21.37	17.64	3.73	Hand Auger
	1C	535279.74	6743753.97	11.96	8.02	3.95	Auger
	2A	535229.52	6743849.19	33.13	23.93	9.20	Washbore
	2C	535284.18	6743843.10	14.62	13.23	1.39	Auger
	3A	535238.17	6743883.30	30.21	24.90	5.31	Washbore
	3B	535264.08	6743882.30	23.87	20.07	3.80	Hand Auger
	3C	535283.81	6743865.97	15.62	13.67	1.95	Auger
Inclinometer	1A	535240.19	6743756.10	29.41	10.355	19.05	Washbore / NMLC Core
	1C	535279.59	6743755.19	12.14	6.394	5.75	Auger
	2A	535230.52	6743850.19	33.18	19.331	13.85	Washbore / NMLC Core
	2C	535284.17	6743842.06	14.67	8.868	5.80	Auger / NMLC Core
	3A	535239.28	6743883.50	30.24	18.942	11.30	Washbore / NMLC Core
	3C	535283.56	6743867.00	15.78	11.533	4.25	Auger

* All piezometers screened from 0.75 m above base, BGL is below ground level,
# Note drilled depth on bore logs is greater than piezometer depth.

Bore logs for each of these locations are provided in Appendix B of WRL Technical Report 2005/34. Department of Natural Resources (DNR) test bore license numbers were allocated as follows:

- Piezometers 1A, 1B and 1C are 30BL183586
- Piezometer 2A is 30BL183587
- Piezometers 3A and 3B are 30BL183588
- Piezometers 3C and 2C are 30BL183589.

A summary of test bore stratigraphy (based on bore logs presented in Appendix B of WRL Technical Report 2005/34), is provided in Table 2. This summary indicates that aquifer geology is dominated by silty sand and that the depth to clay or weathered sandstone varies from 1.7 to 16.4 m below ground. Drilling terminated in sandstone at six of the eight sites.

Table 2
Summary of Test Bore Stratigraphy

Site	Aquifer geology	Depth to clay/ weathered sandstone	Depth to sandstone
		*m bg**	*m bg*
1A	silty sand	16.4	17.4
1B	clayey sand (orange)	3.6	#
1C	sand (grey)	4.2	5
2A	silty sand (grey/iron)	9.2	12
2C	sand to sandy clay (mottled)	3.8	4
3A	sand (grey)	5.2	8.5
3B	sand to silty sand (grey)	3.9	#
3C	silty sand to sandstone	1.7	2.6

*m bg is metres below ground
not drilled to sandstone

3. RAINFALL, WATER LEVEL AND INCLINOMETER MONITORING

Rainfall data was sourced from two locations. Daily rainfall data was provided by Bureau of Meteorology from a pluviometer located at Yamba Pilot Station, on Pilot Hill adjacent to the study area. The Manly Hydraulics Laboratory (MHL) was also contracted by Council to provide rainfall intensity data at 5 minute intervals from a pluviometer also located on Pilot Hill. Records of antecedent rainfall are maintained by Council for the purpose of advising local residents of risk periods for hillslope instability, according to thresholds determined by Jeffrey and Katauskas (2000).

Submersible ODYSSEY pressure transducers and loggers were used to record water levels in each of the piezometers at 30 minute intervals. A WRL barometric transducer and DIVER transducer has also been installed in piezometer 3A to allow barometric corrections of the water level data. Hydrographs for each site are presented in Appendix C.

Vertical land movement is detected with an inclinometer probe which measures changes in tilt of the inclinometer tubing. The method of testing includes lowering the probe to the base of the tubing and taking a reading every 0.5 m as the probe is extracted from the tubing. The tilt at each 0.5 m depth increment is related back to the initial base-line reading, and a plot of movement is produced.

4. RESULTS FROM MONITORING 26TH MAY 2005 TO 25TH MAY 2007

4.1 Rainfall Events

A summary of major rainfall events during the monitoring period is provided in Table 3. Approximately 9 rainfall events (69.3 to 385.1 mm total), each of several days duration occurred during the monitoring period. However, only one event (around 30th June 2005) was rated as a 1 in 10 year ARI (annual recurrence interval) event, for a 2 hour duration. For 18 hours duration, this event was a 1 in 94 year ARI event.

Daily rainfall was plotted with groundwater levels (Figures 4 to 7), and rainfall intensity data (5 minute interval) was used to analyse lag times for groundwater level response. Rainfall intensity during events averaged 17.4 mm/hr, with a maximum of 270 mm/hour (median 10 mm/hour).

Antecedent rainfall levels for warning levels was maintained during the project period by Council. Additional detailed rainfall analysis was carried out specifically for the geotechnical assessment (Section 4.1.2 of Appendix B).

Table 3
Rainfall Events During Monitoring Period

Event No.	Event date	Rainfall daily max (mm)	Total rainfall (mm)	No. consecutive rain days
9	6-Jun-07	27.8	69.3	4
8	9-Nov-06	68.4	122	9
7	30-Aug-06	155.8	215.6	8
6	18-Jun-06	5	128.6	8
5	4-Mar-06	66.4	162.4	10
4	9-Jan-06	62.4	141.9	9
3	24-Nov-05	75.8	100.2	4
2*	1-Jul-05	73.1	385.1	8
1*	30-Jun-05	250.4	385.1	8

* These storms included a 2 hour duration, 1 in 10 yr ARI event, and an 18 hour duration, 1 in 94 yr ARI event.

4.2 Groundwater Levels

Groundwater levels and daily rainfall for Transect 1, 2 and 3 and for MSA monitoring wells on the Pacific Hotel site are presented in Figures 4 to 7. Where no groundwater level is shown during other periods, the piezometer was dry or the water level was below the base of the logger (see Appendix E for details). Logger sensors were located a few centimetres above the base of the piezometers. Due to logger failure, data was not recorded at Site 2A between 6/7/2006 and 13/10/2006 or at Site 2C between 10/10/05 and 11/1/06. The faulty loggers were replaced by GDCS.

A more detailed hydrograph for the major rainfall event around 30th June 2005 is presented with 5 minute rainfall data (Figure 8). The lower piezometers across the study area (i.e. sites B and C) showed greater response to this June 2005 event than the upper piezometers (Table 4). Sites 1A, 2A and 3A showed 0.6 to 0.95 m rise in level over a period of approximately 3 to 7 days. Sites 1B, 1C, 2C, 3B and 3C displayed rises of 1.12 to 1.74 metres over a period of between 12 and 24 hours. However, it is noted that the initial rise rate of piezometer 3A was fast but delayed by unknown processes.

The lower piezometers all began to respond to the event before the upper piezometers. This data reflects the fact that there is a limited catchment area upslope of the study area. As a result, piezometers located near the base of the slope are characterised by a larger response to rainfall infiltration over a larger, upslope area.

Table 4
Groundwater Level Response to Rainfall Event 29th June – 7th July 05

Location	Incipient Levels		Peak Levels		Statistics		
	Water Level Immediately Prior To Response	Date/Time	Maximum Water Level	Date/Time	Magnitude of Rise	Lag	Maximum Rate of Rise
	m AHD		*m AHD*		*m*	*hours*	*mm/hr*
1A	21.38	30/06/2005 9:00	22.13	7/07/2005 17:30	0.75	176.50	21
1B	17.60	30/06/2005 7:30	18.72	30/06/2005 19:30	1.12	12.00	222
1C	8.49	30/06/2005 6:30	10.22	30/06/2005 23:30	1.74	17.00	58
2A	24.41	1/07/2005 0:00	25.05	5/07/2005 20:15	0.63	116.25	18
2C	13.25	29/06/2005 11:15	14.41	30/06/2005 10:00	1.17	22.75	117
3A	25.30	30/06/2005 16:45	26.26	3/07/2005 20:15	0.95	75.50	219
3B	19.80	30/06/2005 6:45	21.11	30/06/2005 16:15	1.31	9.50	265
3C	14.25	29/06/2005 4:00	15.44	30/06/2005 14:15	1.19	34.25	122

* Lag time for each piezometer is the difference between time of first response and time at which peak levels were recorded.

Groundwater level data and cross section for Transect 3 is shown in Figure 9. This illustrates the time lag responses of the piezometers/transects after major rainfall events. The upper plot shows piezometric levels and four time markers. The lower plot shows an elevation cross section of the slope at each transect with piezometric levels at the times shown by the markers.

4.3 Inclinometer Monitoring

Inclinometer monitoring was carried out during the period of 29th April 2005 (initial baseline setup reading) to 8th March 2007 (Section 4.2 of Appendix A).

All inclinometers with the exception of Location 2C indicated little or no significant movement. However, Location 2C showed 5 mm of movement with the plane of movement at a depth of about 3 m. Sandy clays exist at 3 m depth in Borehole 2C, therefore the movement is occurring within the sandy clays.

The mode of movement was consistent with the stability results for Transect 2 (Section 5.4.3 of Appendix A), which showed that the lowest factor of safety occurs for slip circles at the toe of the slope when higher groundwater levels exist during rainfall periods. The movement in inclinometer 2C tends to suggest a slow moving (creep) type movement occurring at the toe of the slope.

Additional elevation surveying is not required after 2 years of inclinometer monitoring since only 5 mm of movement was detected at one of the six sites, compared to elevation survey accuracy of 1-2 cm. Efforts were instead directed at more comprehensive assessment.

5. COASTAL PROCESSES AND HAZARDS

5.1 Coastal Hazards Affecting the Beach

Both extreme rainfall events and beach erosion may be caused by the same meteorological system, that is an "east coast low" or an ex tropical cyclone. MHL (2002a) estimated the following beach hazard design parameters for Yamba Beach:

Typical dune height:	4.0 m
Upper limit storm demand volume:	50 m^3/m
Volumetric loss due to ongoing recession:	0.4 m^3/m/year
[Historical] Linear shoreline recession (all data):	0.1 m/year
Linear recession due to future sea level rise (100 years):	18.1 m
Inundation levels due to wave runup	5.9 m AHD.

5.2 Effect of Stratigraphy and Structures

Both storm erosion and future recession (i.e. long-term processes) will erode sand from the toe of the slopes fronting Yamba Beach, however, much of the beach (particularly the northern end) is fronted by rock cliffs/shelves which would limit short term erosion. This is why the low storm demand volume (of 50 m^3/m) was adopted by MHL (2002a), compared with typical values of 150 to 200 m^3/m adopted for open coast NSW beaches.

Transect 1, if extended seawards (Figure 2) is likely to have rock shelves which limit erosion, but the location of these below approximately 7 m AHD is not known. It is recommended that the rock-sand interface be properly determined. Rock cliffs are evident for other geotechnical Transects.

5.3 Cliff Erosion

Although the rock cliffs and ledges would resist short term beach erosion, when exposed they are likely to suffer from long term weathering and erosion. Detailed studies would need to be undertaken specifically for Yamba Beach. Studies for other NSW sites are summarised below. These reported mean rates need to be interpreted with regard to the episodic nature of cliff collapse events – that is, many years elapse between major events.

Chapman *et al.* (1982) gave the following commentary on cliff erosion in NSW:

Rates of cliff erosion are highly variable and actual measurements are virtually non-existent. …cliff retreat is highly erratic, with localized and infrequent rock falls separated by long periods of weathering.

For the purposes of estimating sediment supply to beaches, Chapman *et al.* (1982) suggested an order of magnitude estimate of cliff erosion rates for Sydney to be 5 mm per year.

Sunamura (1983) presented a model for cliff recession and collated recession rates from numerous locations around the world. The only Australian locations cited were for limestone at Point Peron near Perth (0.2 to 1 mm per year) and aeolianite at Warrnambool Victoria (14 mm per year). Sunamura also presented results of physical model studies on cliff recession and platform formation.

Dragovich (2000) estimated erosion rates of Sydney sandstone in locations with a high salt load to be 1 to 5 mm per year – though this related to dimensioned construction stone rather than sea cliffs subject to wave action. Dragovich (2000) also quoted Roy (1983) who estimated that the softer beds near the base of sandstone cliffs in the southern Sydney region were weathering at rates of 2 to 5 mm per year.

5.4 Beach Amenity and Cultural Heritage

In addition to the risk to public safety, a landslip event could potentially reduce useable beach width, block public beach access for an extended duration and/or damage the historic Yamba Surf Lifesaving Clubhouse.

6. HYDROGEOLOGICAL ASSESSMENT

6.1 Conceptual Model

The observed behaviour of groundwater levels in response to rainfall during the monitoring period was consistent with the initial conceptual hydrogeological model (Jeffrey and Katauskas, 2000). Detailed analysis of borehole stratigraphy and groundwater monitoring data was used to verify and refine the hydrogeological model shown in Figure 10.

Groundwater parameters of relevance to slope stability assessment are summarised in Table 5, comparing the initial model with observations from the site. Observed saturated thickness, depth to groundwater, groundwater level rise, lag times and drainage times were all within the range of values allowed for by the initial hydrogeological model.

In summary, observed conditions were consistent with the simplified hydrogeological model, and observed groundwater level response (0.75 to 1.74 m) to rainfall events were within the range (1 to 2 m) that was predicted. The hydrogeological model that was adopted is therefore considered to be conservative and appropriate for the Yamba Hill site.

However, the hydrogeological model (Figure 10) could be improved by allowing for a lower rather than higher lateral hydraulic gradient after rainfall events. A lower hydraulic gradient suggests that the toe of the slope becomes saturated during rainfall events. The hydrographs near the toe of slope had two distinctive recovery rates – early rapid decline (i.e. 77 mm/day) then slower decline (i.e. 6 mm/day) while the average drainage rate was close to the 19 mm/day in the initial conceptual model. The two stage drainage curves were attributed to local drainage immediately after the rain period, followed by a slower groundwater level decline due to additional infiltration from further up the slope arriving at the toe of the slope.

6.2 Lag Times and Groundwater Flow

A detailed assessment of groundwater level response was completed for the largest event during the monitoring period (June 2005, 1 in 94 year ARI, up to 18 hours duration). The most rapid groundwater level response to rainfall event occurred at mid-slope (~10 to 12 hours), followed by groundwater level response at the toe of slope (17 to 34 hours). There was a considerable lag before groundwater levels responded to rainfall at the top of the slope (76 to 177 hours). Piezometer 1A has a much longer response time than other monitoring sites at the top of the slope. Since the travel distance for recharge is about the same, it appears the recharge is inhibited by impermeable cover, although the final magnitude of recharge is similar across the tope of the slope.

Table 5
Summary of Conceptual Groundwater Models

Parameter	**Initial model (J&K 2000)**	**Revised model (June 2005 event)**
Groundwater level rise (m)	1 to 2	0.75 to 1.74
Saturated zone above silty clay or bedrock (m)	0.5 to 1 m	0.3 to 3.5
Depth of sand at crest of slope (m)	3	3.8 to 16.4
Depth of sand at toe of slope (m)	1 to 2	3.8 to 4.2
Depth to clay (m)	9 to 10	1.7 to 16.4
Thickness of clay (m)	2 to 4	0.2 to 3.3
Vertical flow to crest of slope – time lag between rainfall and groundwater level rise (days)	3	3 to 7.4
Vertical flow to toe of slope – Time lag between rainfall and groundwater level rise (days)	0.7	0.7 to 1.4
Lateral flow - time lag between rainfall and groundwater level rise (days)	8 to 40*	11 to 67 (Piezo 1C)
Time to drain after event (days)	10 to 80 (av. 19 mm/day)	96 (Piezo 1C)
Average rise in groundwater levels for 100 mm event (mm)	375[#] (or 300 to 400 mm)	286

* assuming K lateral is 2 to 10 times vertical K
[#] assumes 75% average permeable area and 0.2 effective porosity

Darcy's Law which describes the relationship between groundwater flow, hydraulic gradient and hydraulic conductivity (Equation 1).

$$Q_h = A.dh/dL.\ K_{sat\text{-}h} \qquad (1)$$

Where Q_h is lateral flow rate (m^3/s), A is area (nominally 1 m^2), dh/dL is the hydraulic gradient (m/m) and $K_{sat\text{-}h}$ is saturated horizontal hydraulic conductivity (m/s).

Lateral groundwater flow rates below Yamba Hill were calculated by applying Equation 1. The hydraulic head driving flow through the slope decreased following a rainfall event because groundwater levels rise more at the base than the top of the slope. This groundwater level behaviour differs from that assumed in the initial conceptual model (Jeffrey & Katauskas, 2000).

As discussed in Section 6.1, groundwater behaviour was observed as follows:

- Rapid response to rainfall event
  - decreased dh/dL between A and C (as per Equation 1), therefore, decreased flow
  - increased dh/dL between C and cliff top, therefore increased flow.
- Delayed response to rainfall event
  - increased dh/dL between A and C, therefore, increased flow.

Lateral groundwater flow rates for Transect 1 were calculated to be 0.18 m^3/day (Table 6), similar to the 0.2 m^3/day projected by the hydrogeological model (Figure 10). These flow rates assumed a saturated hydraulic conductivity value of 10^{-5} m/s or 36 mm/hr (Douglas Partners, 1996). These values are considered to be realistic for silty sand, and are unlikely to be improved by site specific testing. It is noted that infiltration of rainfall is determined by vertical hydraulic conductivity of semi-saturated sands (i.e. below the soil zone), which may be 1-2 orders of magnitude less than saturated hydraulic conductivity.

Rates of groundwater drainage for Site 1C are presented in Table 7. Limited drainage rates mean that groundwater levels below the crest of the dune remain high for 2 to 3 months after a major rainfall event. Therefore, antecedent rainfall will be the most critical element in producing groundwater levels which may cause instability of the slope.

Table 6
Lateral Groundwater Flow (Transect 1)

	Infiltration along the transect (m3/hr)			**Total infiltration**		**Hydraulic gradient**	**Groundwater Flow**
Infiltration rate (mm/hr)	**Part 1*** **(20 m)**	**Part 2 (24.5 m)**	**Part 3 (25.5 m)**	**Total (m^3/hr)**	**(m^3/day)**	**m/m**	**m^3/day**
36	0.72	0.882	0.918	2.52	0.105	0.21	0.18

*Parts of the transect as indicated by Figure 8

Table 7
Rate of groundwater drainage (Site 1C, Event 1-2)

	Period of groundwater level decline		
	1st period	**2nd period**	**Total**
Date start	3-Jul-05	1-Aug-05	3-Jul-05
Date finish	14-Jul-05	7-Oct-05	7-Oct-05
Days	11	67	96
SWL decline (m)	0.85	0.43	1.6
mm/day	77.2	6.3	16.3

6.3 Groundwater Level Response at the Pacific Hotel Sites (MSA Monitoring Wells)

Groundwater level responses at existing monitoring sites in the Yamba Hotel area were distinctly different from the new monitoring transects. Response in groundwater levels observed in MSA1 and MSA9 occurred rapidly after rainfall events, possibly due to long intake screens. Intake screens were observed in these existing monitoring bores up to ground level, rather than a short intake screen within the silty sand. Groundwater level recovery (i.e. declines) in MSA9 also appeared to be distinctive to other sites, and may be attributed to the drainage system installed in the mid to lower slope at this site (Section 4.1.1 of Appendix A).

6.4 Recharge Coefficients

Calculated recharge coefficients, while not of direct relevance to geotechnical analysis, were considered as part of the verification process for the hydrogeological conceptual model (Figure 10). The proportion of rainfall that recharges groundwater depends on many factors including land surface/vegetation type, surface slope, depth to groundwater, and storage capacity or porosity of the unsaturated zone. Recharge coefficients, the proportion of rainfall that reaches the water table, were estimated for Yamba Hill transects 1 and 2, located at the base of slope. Median and average rainfall intensity for the monitoring period was 10 and 17.4 mm/hr, less than the 36 mm/hr maximum infiltration rate assumed for unsaturated sandy materials.

Average recharge coefficients for Transects 1 and 2 were 42% and 34% respectively, assuming a porosity of 0.2 for silty sand (Tables 8 and 9). Average recharge coefficients were based on at least 3 major rainfall events. The total recharge along the transect assumed 60% of Transect 1 was permeable and 44% of Transect 2 was permeable, with the remainder of the transect impervious surfaces such as pavement and roofing.

The range of recharge coefficients, and values greater than 100% reflected the uncertainty in assumed porosity values and permeable sections of the Transects. Low or high anomalous values were not included in average estimates (Table 8). These estimated recharge coefficients were somewhat higher than the 30% value often assumed for groundwater management in coastal sand aquifers. However, considering the possible range in porosity values, the recharge coefficients are considered to broadly consistent with expected hydraulic behaviour of these types of sediments.

Table 8
Recharge Response from Various Rainfall Events (Site 1C)

Event No.	SWL rise m	SWL rise mm	Event date	Daily max (mm)	Total (mm)	No. consecutive rain days	Recharge (mm)	% of rain
1	1.74	1740	30-Jun-05	250.4	385.1	8	208.8	54.2
3	0.864	864	24-Nov-05	75.8	100.2	4	103.7	103.5
6	0.287	287	18-Jun-06	5	128.6	8	34.4	26.8
7	0.813	813	30-Aug-06	155.8	215.6	8	97.6	45.3
8	0.046	46	9-Nov-06	68.4	122	9	5.5	4.5
							Average	**42.1***

* Average of Event 1, 6 and 7. Assumes porosity of 0.2, and 60% of transect is permeable.

Table 9
Recharge Response from Various Rainfall Events (Site 2C)

Event No.	SWL rise m	SWL rise mm	Event date	Daily max (mm)	Total (mm)	No. consecutive rain days	Recharge (mm)	% of rain
1	1.17	1170	30-Jun-05	250.4	385.1	8	103.0	26.7
5	0.575	575	4-Mar-06	66.4	162.4	10	50.6	31.2
6	0.56	560	18-Jun-06	5	128.6	8	49.3	38.3
7	0.901	901	30-Aug-06	155.8	215.6	8	79.3	36.8
8	0.506	506	9-Nov-06	68.4	122	9	44.5	36.5
							Average	**33.9**

* Average of Events 1 and 5-8. Assumes porosity of 0.2, and 44% of transect is permeable.

7. SUMMARY OF GEOTECHNICAL ASSESSMENT (APPENDIX A)

The following sections are summarised from the report by Jeffery and Katauskas (2007) that is presented in Appendix A.

7.1 Geotechnical Stability Analysis

The slope stability analysis considered surface conditions, sub-surface conditions, including suitable material properties (eg. effective cohesion, effective friction angle), together with groundwater level response to rainfall. A computer program SLOPE/W was used to analyse slope stability by considering circular failures through the sandy silt overlying bedrock. Although circular failure is not always the case, it is considered to be a reasonable approximation for many failures.

A factor of safety (FOS) is used for traditional stability analysis as follows:

FOS 1	=	incipient instability
FOS > 1	=	failure should not occur
FOS < 1	=	failures should have occurred.

Overall the stability analysis of three subsurface models (Transects 1, 2 and 3) showed the slopes have low factors of safety (FOS) particularly for higher groundwater levels and the slope close to the Pacific Hotel. Calculated FOS values ranged from 1.0 to 1.6 for varying locations, types of failures and groundwater level conditions. These FOS values were generally less (i.e. higher risk) than the usually accepted values of at least 1.5 for "reasonable design case" and as low as 1.25 that may be tolerated for transient short term conditions.

7.2 Landslide Risk Analysis

The risk analysis included rainfall analysis and probability assessment, in accordance with the AGS (2000) Risk Management Guidelines. The earliest known landslide occurred in May 1938, with several recorded events since then. A historical search for landslide events identified 11 dates between 1921 and 1999 (Jeffrey and Katauskas, 2000). Landslide events were categorised as:

- Scour – high intensity rainfall
- Earthslides
- Earthflows.

There were no landslides in the study area during the monitoring period for this project.

The probability of a landslide occurring was estimated by J&K to be as follows:

- For earthslides and scour at the toe of the slope – 1 in 10 years to 1 in 125 years. However, considering that a landslide occurs only 50% of the time a 'trigger' level is reached, the probability equates to 5×10^{-2} to 4×10^{-3} (or 1 in 20 years to 1 in 250 years).
- For earthslides encompassing the steeper hillslide slopes – 1 in 10 years to 1 in 100 years. For a 50% trigger, this probability equates to 5×10^{-2} to 5×10^{-3} (or 1 in 20 years to 1 in 200 years).

Risk was then determined as a function of probability and consequence. Risk estimates were determined in relation to the suggested criteria in AGS (2000), with 10^{-4} tolerable risk and 10^{-5} as acceptable risk for loss of life of person most at risk. It will be up to the owners to decide whether these values are appropriate and the conclusions regarding the risk estimates reasonable.

The highest risk values identified were associated with Landslide Risk Zone 1a (LRZ1a, Figure 11). This zone was characterised by steepest slopes, a history of movement and expected high occupancy rate. In this zone the results of the risk assessments were:

- For slow to very slow movement, probability = 5×10^{-5} (tolerable)
- For rapid to very rapid movement, probability = 10^{-3} (unacceptable).

For LRZ1b which includes residential dwellings to the north of the Pacific Hotel the risk assessments were:

- For slow to very slow movement, probability = 10^{-5} (acceptable, just)
- For rapid to very rapid movement, probability = 4×10^{-4} (unacceptable).

The data obtained from investigations and monitoring during this project do not allow any adjustment to the LRZs.

8. RECOMMENDATIONS

These recommendations confirm and expand upon interim recommendations provided to Clarence Valley Council by email 2nd April 2007 and WRL Letter 24th April 2007.

On the basis of the revised assessment, it is considered that emergency levels and management strategies that were put into place in October 2000 should remain in place until more permanent stabilisation measures are adopted. Two warning levels were set up as an interim measure (Jeffrey and Katauskas 2000) - an Orange level which was based on a 1 in 3 year rainfall and a Red level which was based on a 1 in 10 year rainfall, taking into account antecedent rainfall over periods of 1 to 90 days.

Various slope treatment/stabilisation options should be investigated in more details with a view to implementation as a matter of priority.

Groundwater monitoring should continue in the same method and frequency used in this study. The specialist groundwater and barometric pressure loggers deployed at site 3A by WRL has been replaced by a standard groundwater level logger serviced by GDCS.

Inclinometer measurements may be extended to an annual basis unless significant rainfall events occur and/or movements of the slope are observed. In such a case, the inclinometers should be read as soon as possible.

An assessment of groundwater levels and slope stability should be undertaken if a significant rainfall event occurs. It is recommended that a significant event should not be limited to single day events and should include antecedent events. In particular, any longer term antecedent events that result in a red zone management alert should be addressed.

9. REFERENCES

AGS (2000), Landslide Risk Management Concepts and Guidelines. Australian Geomechanics Society Sub-Committee on Landslide Risk Management.

Chapman, D. M, Geary, M, Roy, P. S. and Thom B. G. (1982), Coastal Evolution and Coastal Erosion in New South Wales, a report prepared for the Coastal Council of New South Wales, Sydney, ISBN 0 7240 6582 2.

CVC (2004), Yamba Coastline Management Plan – Implementation Strategy. Issued 16 March, 2004. Clarence Valley Council

Douglas Partners (1996), Phase One Geotechnical Investigation, Marine Parade Restoration, Yamba, New South Wales. Prepared for Maclean Shire Council, Project No. 17632A.

Dragovich, D. (2000), "Weathering Mechanisms and Rates of Decay of Sydney Dimension Sandstone", in *Sandstone City*, edited by GH McNally and BJ Franklin, *Monograph No. 5*, Geological Society of Australia, pp. 74-82.

Jeffrey and Katauskas Pty Ltd (2000), Report to Manly Hydraulics Laboratory NSW Department of Public Works and Services on Geotechnical Assessment for Yamba Coastline Management Study.

Jeffrey and Katauskas Pty Ltd (2007), Report to Water Research Laboratory, Updated Geotechnical Assessment for Pilot Hill, Yamba.

MHL (2002a), Yamba Coastline Management Study - Stages 1 & 2 Coastal Processes and Hazard Definition. Report MHL1045, May 2002.

MHL (2002b), Yamba Coastline Management Study - Stage 3 - Management Options

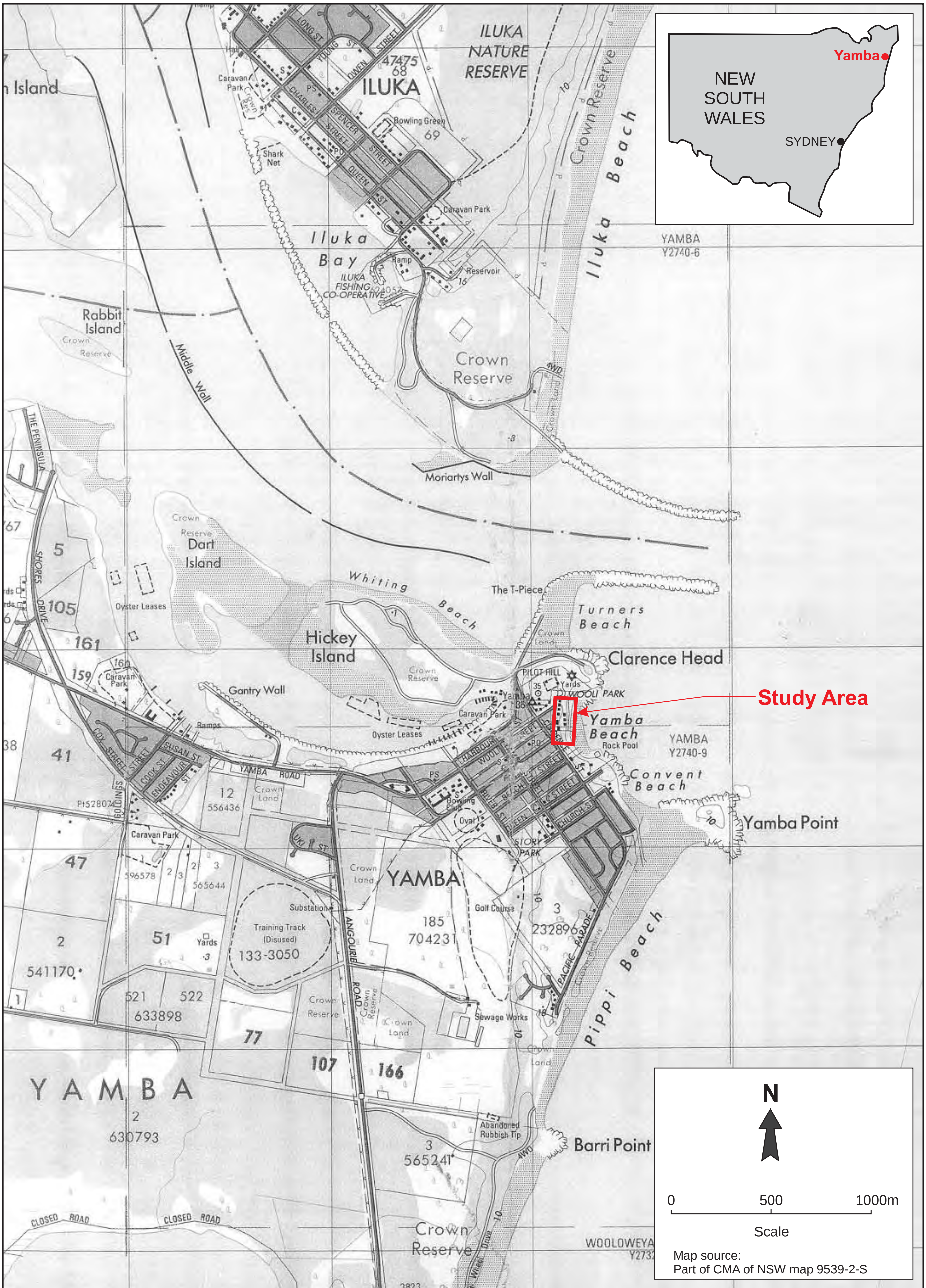
MHL (2003), Yamba Coastline Management Study - Stage 4 - Draft Coastline Management Plan. Report MHL1142, June 2003.

Roy, P. S. (1983), “Cliff Erosion Rates in the South Sydney Region, Central New South Wales Coast” *Geological Survey of NSW Quarterly Notes 50, pp 8-11.*

Sunamura, T (1983), “Processes of Sea Cliff and Platform Erosion”, in *CRC Handbook of Coastal Processes and Erosion*, Edited by Komar, PD and Moore JR, CRC Press, Boca Raton, Florida, USA.

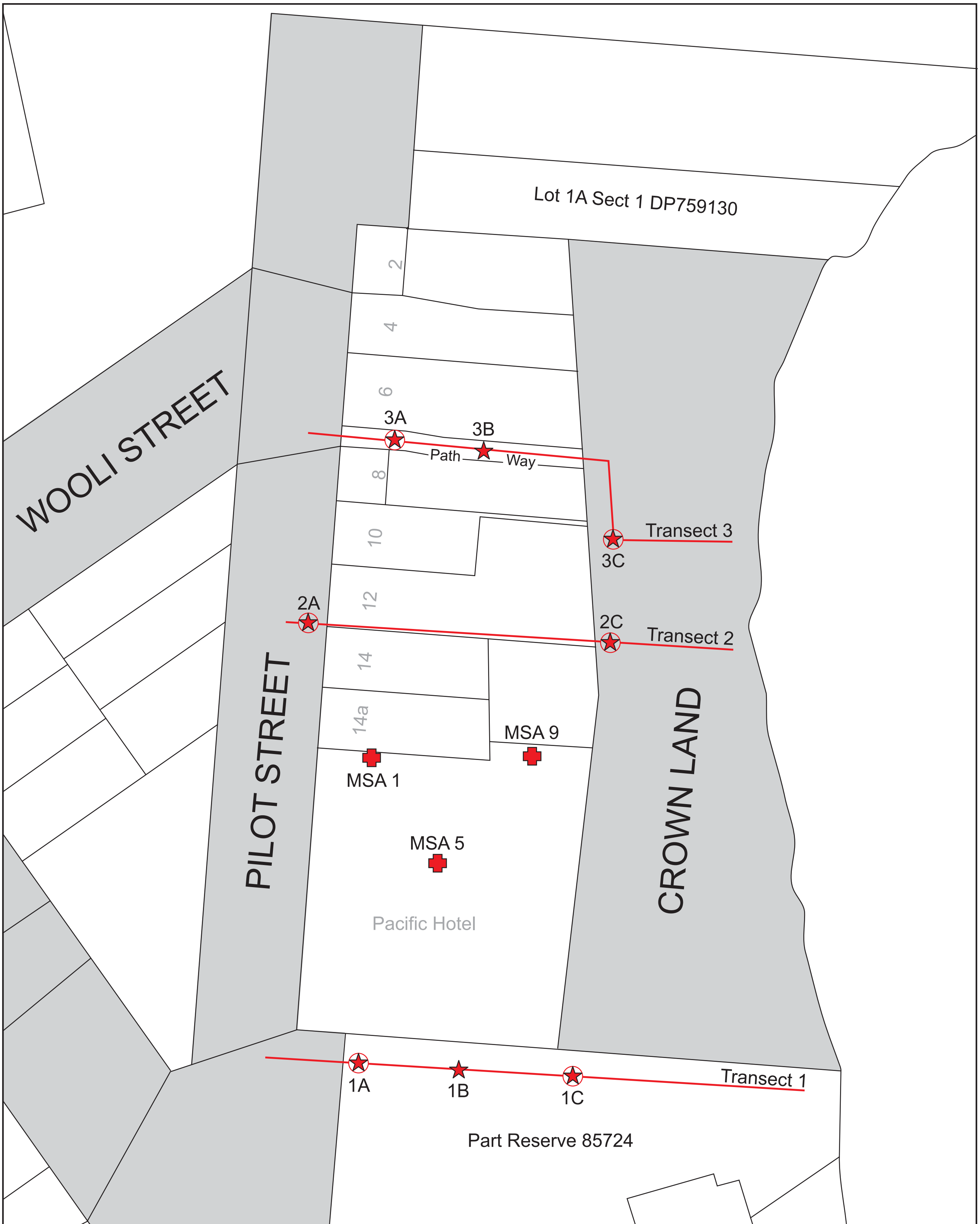
Timms, W. A., Pells, S. E. and Cunningham, I. L. (2005), Yamba Hill Groundwater Level Monitoring – Interim Report. UNSW Water Research Laboratory Technical Report 2005/34.

Young, R. W. and Wray R. A. L. (2000), "The Geomorphology of Sandstone in the Sydney Region", in *Sandstone City*, edited by GH McNally and BJ Franklin, *Monograph No. 5*, Geological Society of Australia, pp. 55-73.



WRL Report No. 2007/32	LOCATION MAP	Figure 1

04154-01.cdr



Map courtesy of Clarence Valley Council

LEGEND:
- Inclinometer
- Piezometer
- Existing monitoring site

WRL Report No. 2007/32	LOCATIONS OF PIEZOMETERS, INCLINOMETERS AND TRANSECTS 1 - 3	Figure 2

04154-02.cdr

A

B

WRL Report No.2007/32	PHOTOGRAPHS OF YAMBA HILL SITE	Figure 3

WRL
Report No.2007/32

GROUNDWATER LEVELS AND DAILY RAINFALL – 26TH MAY 2005 TO 25TH MAY 2007 – TRANSECT 1

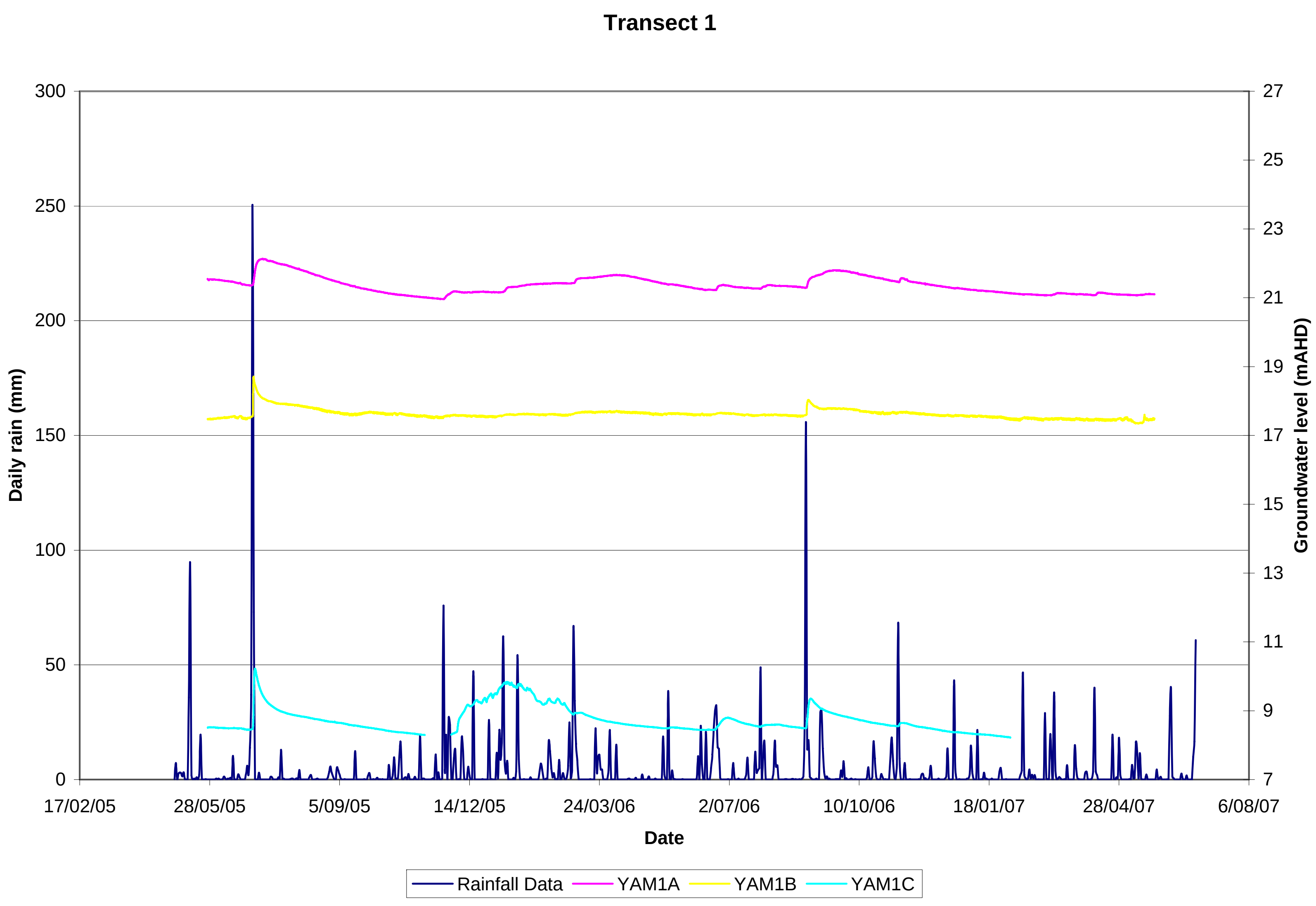
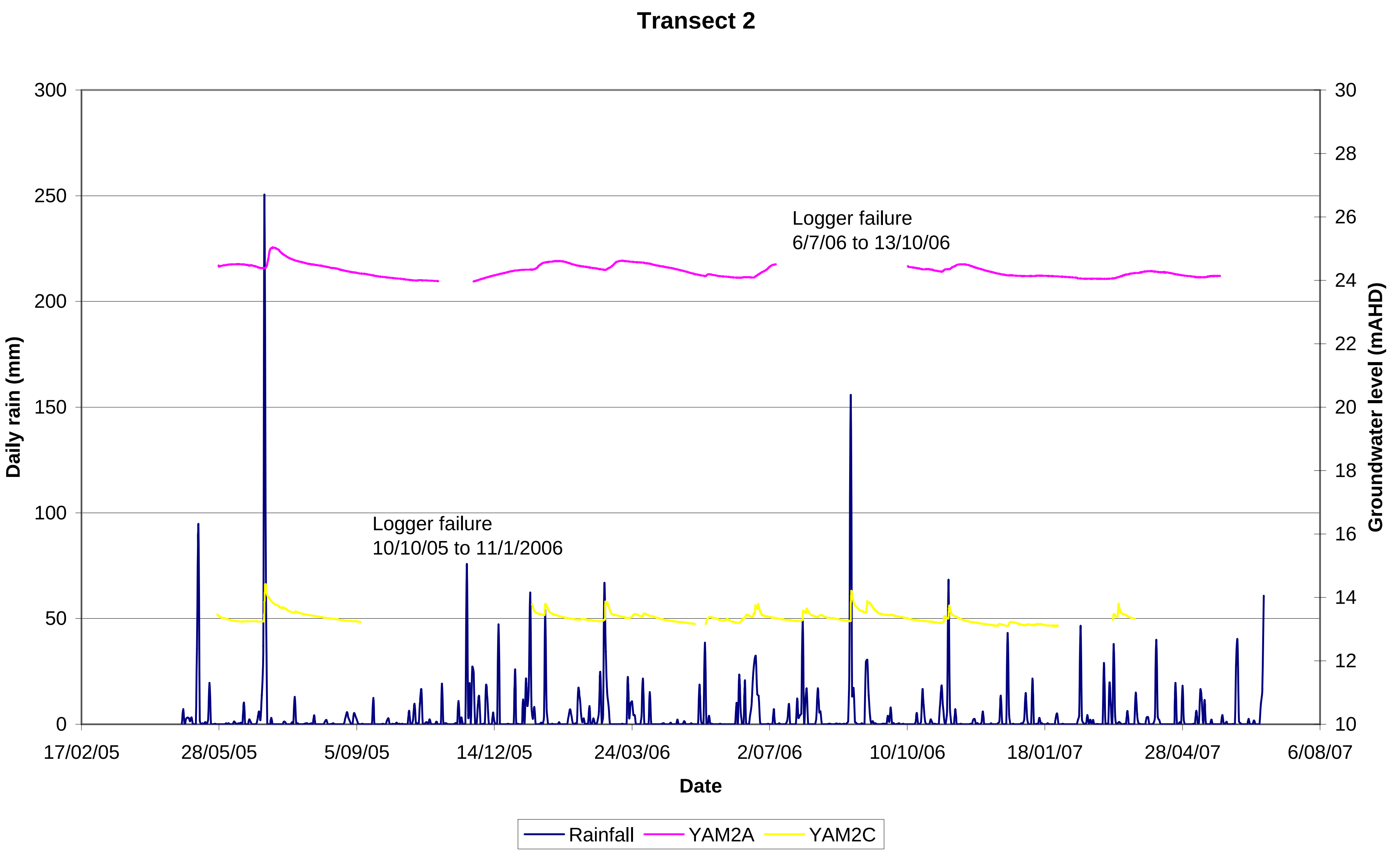


Figure
4



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Report No.2007/32

GROUNDWATER LEVELS AND DAILY RAINFALL – 26TH MAY 2005 TO 25TH MAY 2007 – TRANSECT 2

Figure 5

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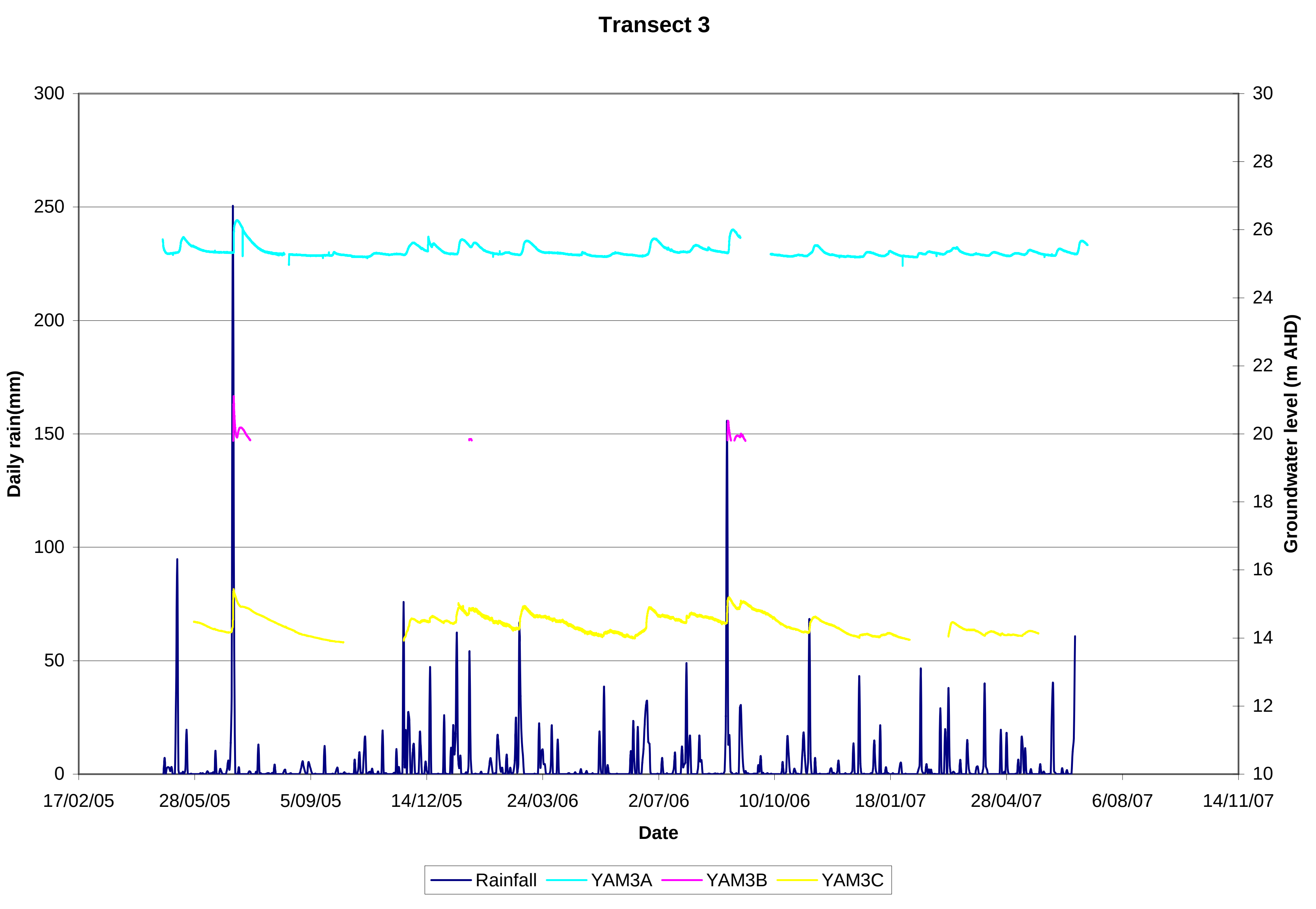
GROUNDWATER LEVELS AND DAILY RAINFALL – 26TH MAY 2005 TO 25TH MAY 2007 – TRANSECT 3

Figure 6



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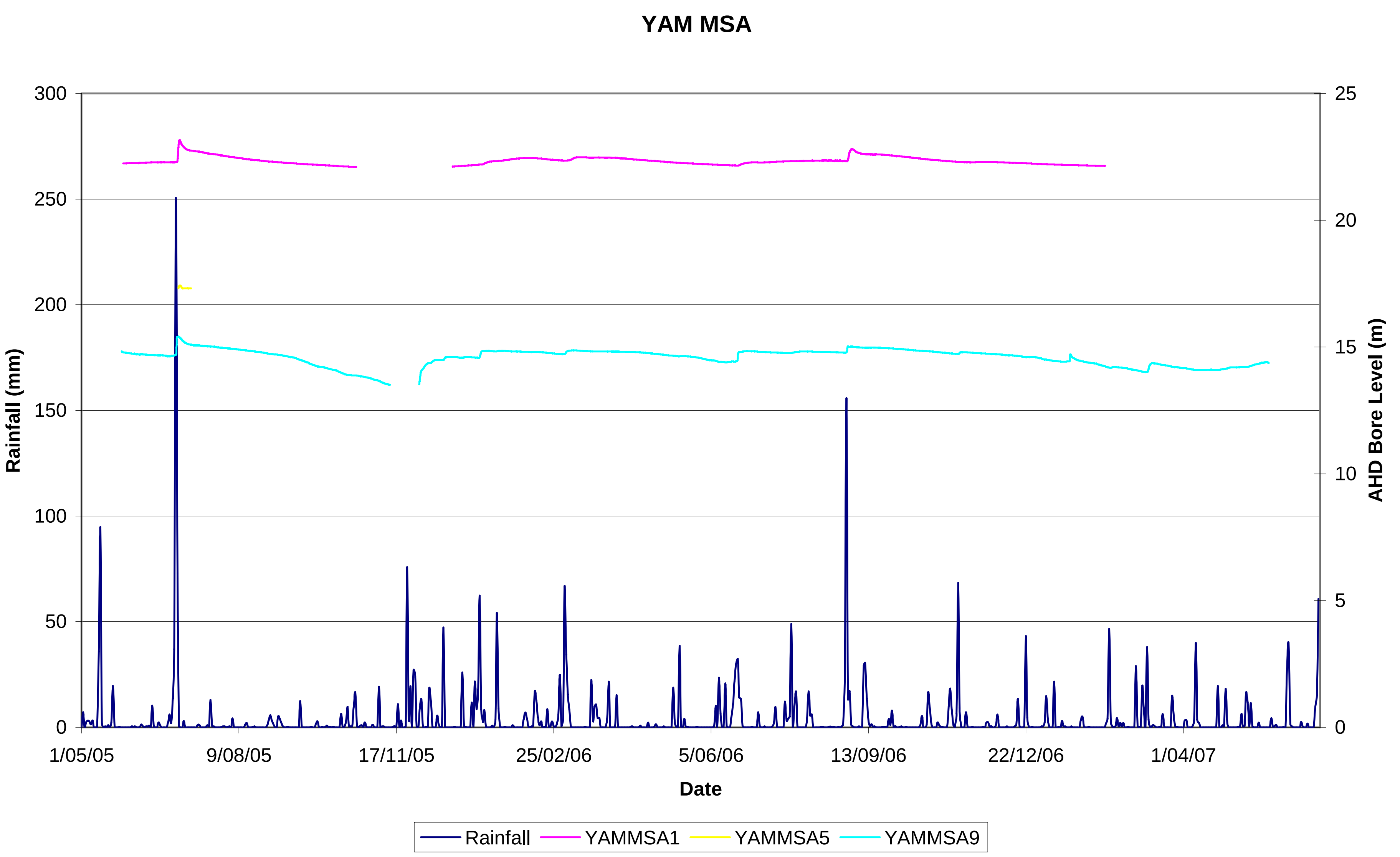
Report No.2007/32

GROUNDWATER LEVELS AND DAILY RAINFALL – 26TH MAY 2005 TO 25TH MAY 2007 – MSA SITES

Figure 7



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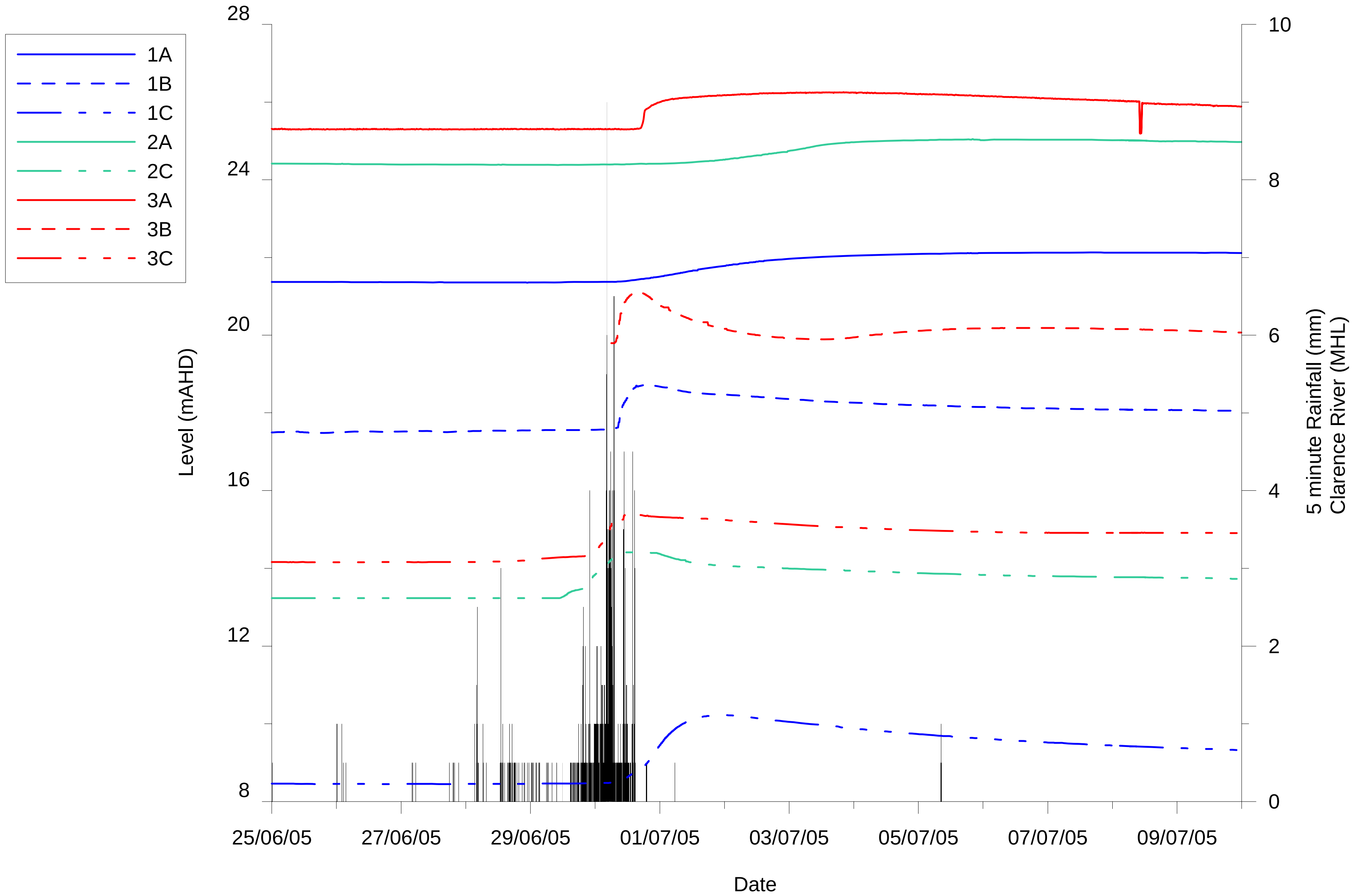
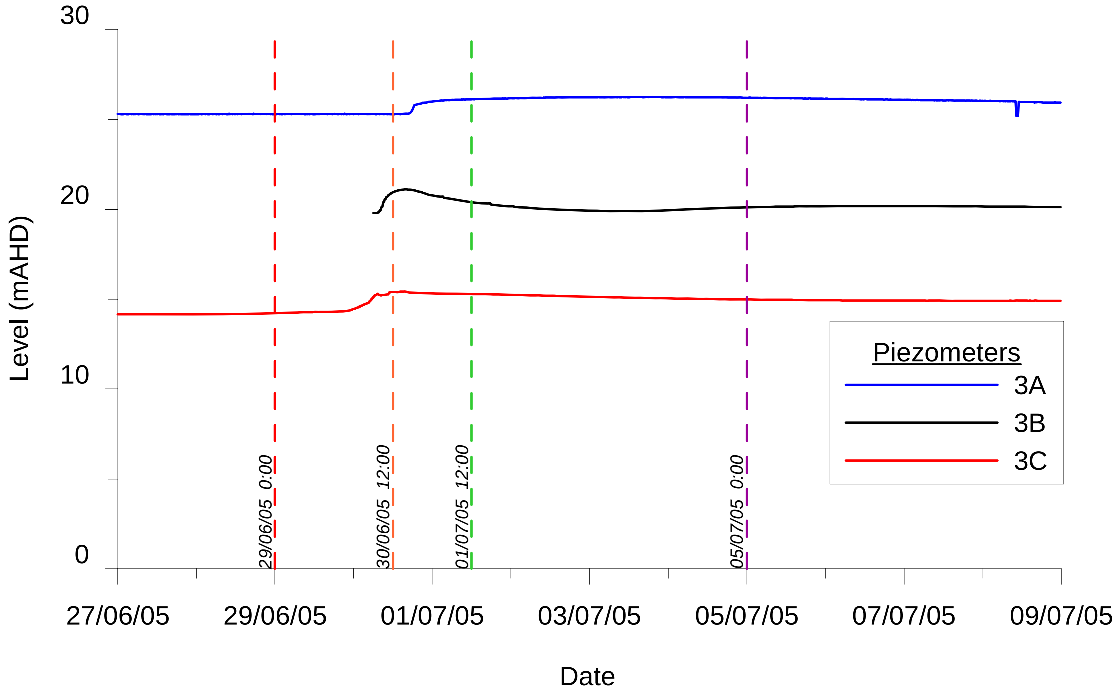
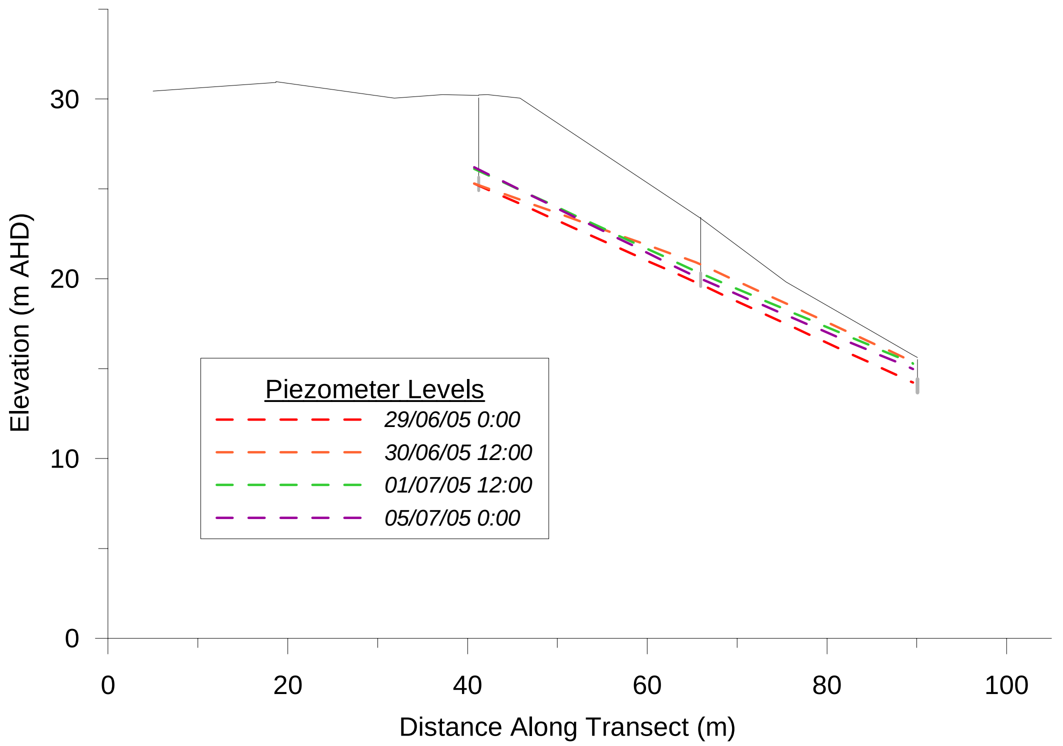


RAINFALL (mm/5min) AND GROUNDWATER LEVELS (mAHD) 25 JUNE TO 10TH JULY 2005

Figure 8



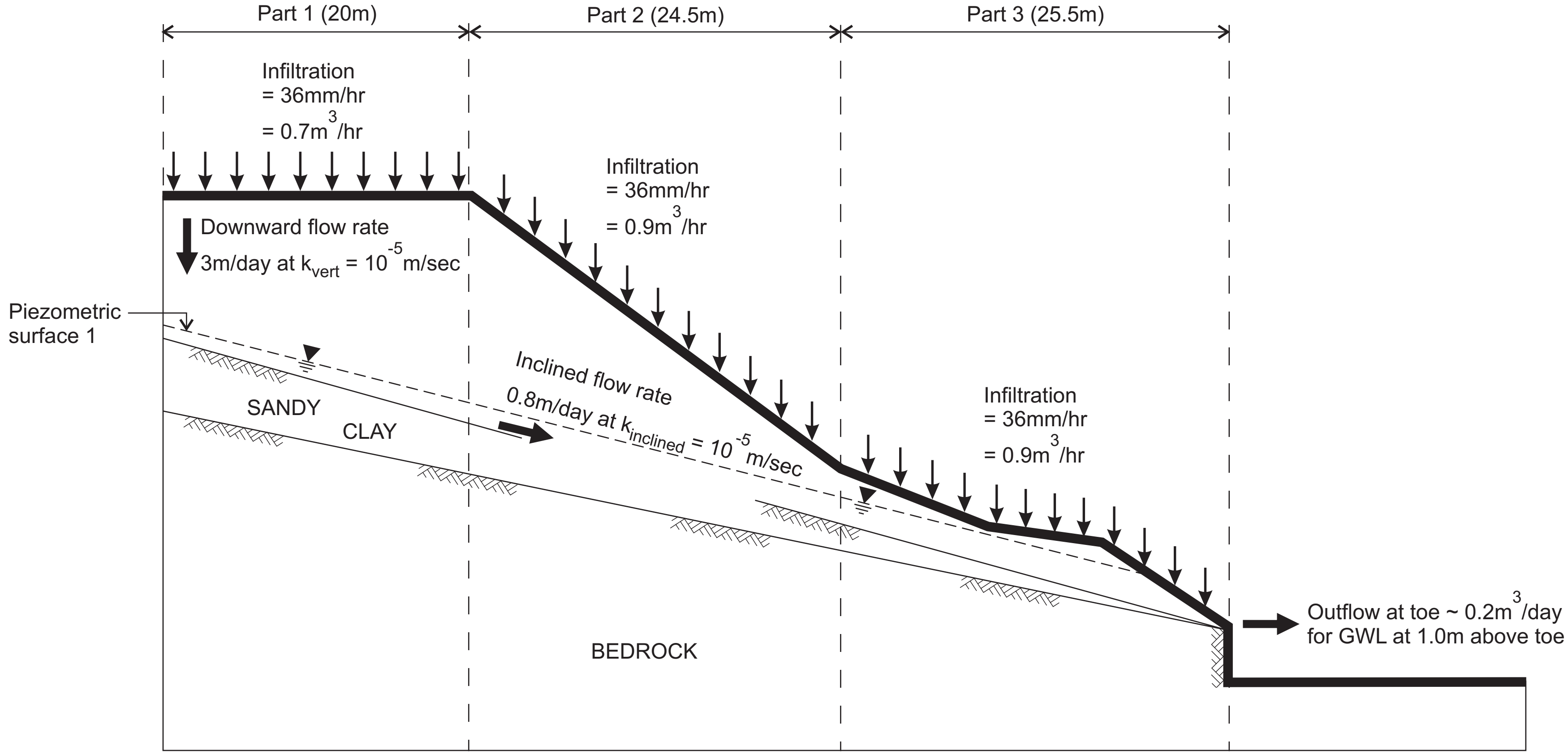


WRL Report No.2007/32	GROUNDWATER LEVELS 29TH JUNE TO 5TH JULY 2005 AND SECTION FOR TRANSECT 3	Figure 9

WRL

Report No. 2007/32

HYDROGEOLOGICAL MODEL

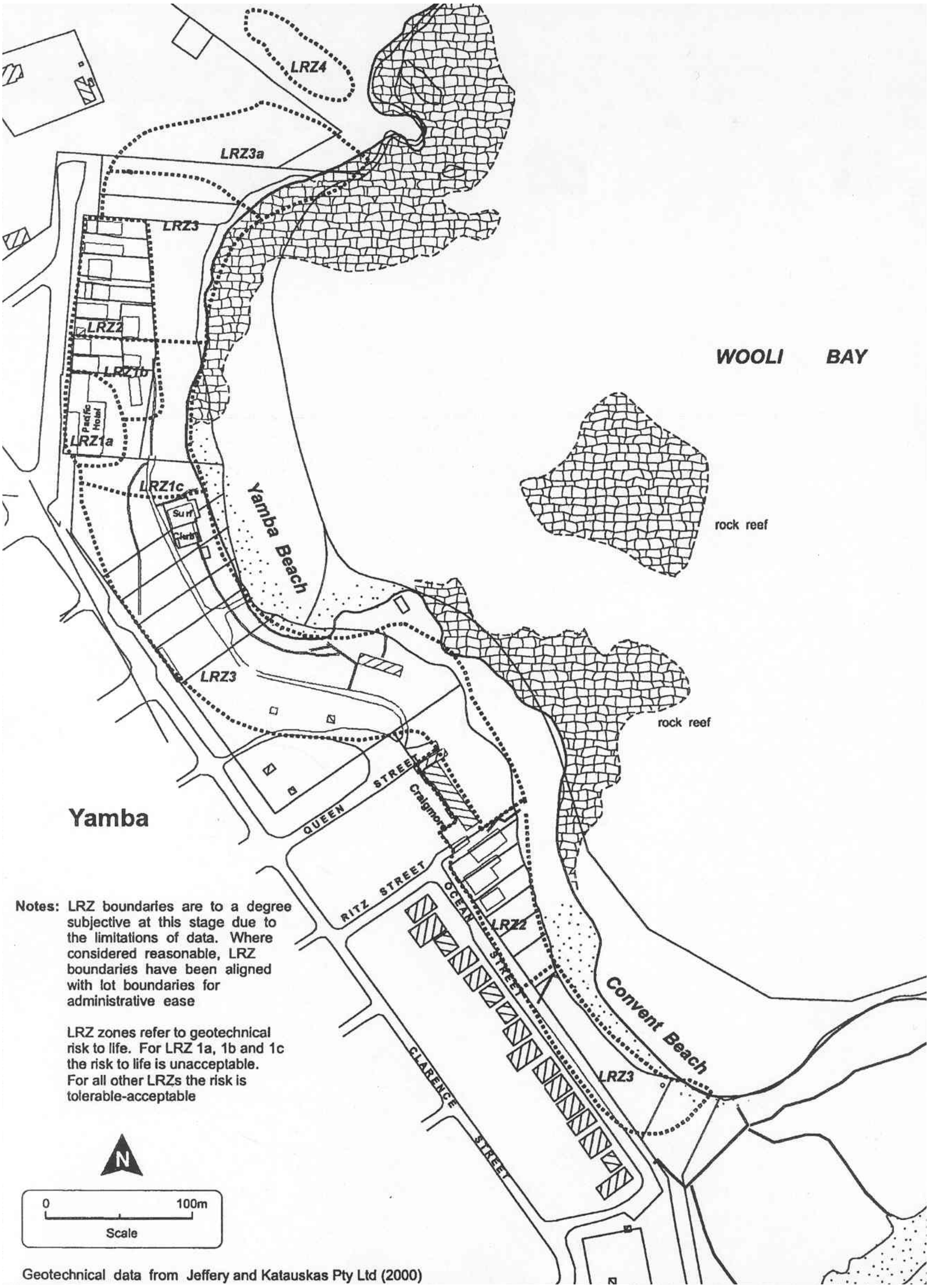


NOTES:

- Infiltration volumes are per unit width.
- Approximate time to flow from centre of part 1 to outflow at toe = 60m x 0.8m/day = 75 days.
- Inclined flow may vary by an order of magnitude or more due to layering within the subsurface soil.
- Revised model based on J & K (2000).

Figure 10

04154-10.cdr


LRZ4
LRZ3a
LRZ3
LRZ2
LRZ1b
Pacific Hotel
LRZ1a
LRZ1c
Surf Club
Yamba Beach
WOOLI BAY
rock reef
rock reef
LRZ3
Yamba
QUEEN STREET
Craigmore
RITZ STREET
OCEAN STREET
LRZ2
Convent Beach
LRZ3
CLARENCE STREET
Notes: LRZ boundaries are to a degree subjective at this stage due to the limitations of data. Where considered reasonable, LRZ boundaries have been aligned with lot boundaries for administrative ease
LRZ zones refer to geotechnical risk to life. For LRZ 1a, 1b and 1c the risk to life is unacceptable. For all other LRZs the risk is tolerable-acceptable
N
0
100m
Scale
Geotechnical data from Jeffery and Katauskas Pty Ltd (2000)

APPENDIX A

UPDATED GEOTECHNICAL ASSESSMENT BY JEFFERY AND KATAUSKAS PTY LTD (11TH SEPTEMBER 2007)

REPORT

TO

WATER RESEARCH LABORATORY
SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING
UNIVERSITY OF NEW SOUTH WALES

ON

UPDATED GEOTECHNICAL ASSESSMENT

FOR

PILOT HILL YAMBA

Date 13 September 2007

Ref:19314WLrpt2

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

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1 INTRODUCTION

This report presents the results of our revised geotechnical assessment of slope instability for Pilot Hill Yamba. The assessment has been carried out following receipt of a letter of commission dated 7 March 2005 from The University of New South Wales Water Research Laboratory (WRL).

The purpose of our revised geotechnical assessment was to utilise direct groundwater monitoring, rainfall data and inclinometer measurements over a two year period (May 2005 to June 2007) to revise probability estimates and risk ratings for landsliding in the Pilot Hill area. It is beyond the scope of this assessment to provide any detailed stabilisation designs.

Water Research Laboratory (WRL) have directed the field investigations, groundwater monitoring and analysis and have provided the data to us for our geotechnical assessment. Other processes affecting the study area, such as beach erosion and shoreline recession are being separately addressed by WRL. Therefore this report should be read in conjunction with the WRL Technical Report Reference 2007-32 dated September 2007 (WRL TR 2007-32)

1.1 Current Study Area

The current study area comprises Yamba Beach which is backed by steep foreshore slopes leading up to Pilot Street which is located on a plateau area. The Pacific Hotel and various residential dwellings are located toward the crest of the foreshore slopes. The attached Figure 1 shows a general location map, while Figure 2 shows a more detailed location plan including investigation locations. These figures are extracts from the WRL report and have been included in our report for completeness. This current study area was chosen as it represented the highest risk to life of the areas addressed in our previous report in October 2000. It therefore warranted more detailed analysis of the groundwater and subsurface conditions in order to assess the risks in relation to actual site data rather than broad conceptual models and to enable more specific site data for any future stabilisation works.

2 HISTORY

2.1 Previous Geotechnical Assessment

Jeffery and Katauskas completed a previous geotechnical assessment as part of the Yamba Coastline Management Study (Report Reference14989WL, dated 17 October 2000). At the time of completing the previous report there was no groundwater monitoring and only limited subsurface data for the Pilot Hill site. The previous report therefore considered a broad conceptual geotechnical model incorporating available geological data and hydro-geological information. Consideration was given to the history of landsliding which was related to a statistical analysis of rainfall data. The rainfall return period for known landslides was used to derive the probability of instability. The issue of slope instability was addressed in a quantitative risk management format together with traditional analysis of the factor of safety. The quantitative risk assessment was carried out for risk to life within a number of zones of similar character. As a result of that assessment, options for stabilisation remediation measures were identified. We recommend this current revised assessment be read in conjunction with that previous assessment.

2.2 Subsurface Investigations and Installation of Monitoring Wells by Others

Since our geotechnical assessment in October 2000, Michael Samms and Associates have undertaken additional subsurface investigations presented in a factual report (Reference 99-629, dated 24 June 2001). The subsurface investigations included drilling eight boreholes and installation of nine monitoring wells (MSA1 to MSA9 inclusive, with MSA9 installed in a previous Douglas Partners borehole). Plots of groundwater levels in the monitoring wells verses time for the period from 10 October 2000 to 13 April 2001 were prepared. The nine monitoring wells were installed in an area below (to the east) of the northern half of the existing Pacific Hotel building. We note that three of these monitoring wells (MSA1, MSA5 and MSA9) have been utilised as part of this current geotechnical assessment by placing groundwater monitoring data loggers within the monitoring wells. These monitoring locations have also been nominated as YAMMSA1, YAMMSA5 and YAMMSA9 in the WRL report.

2.3 Installation of Horizontal Sub-Soil Drains

Since completion of our previous geotechnical assessment in October 2000, we understand that three horizontal sub-soil drains have been installed within the Pacific Hotel site, in and around the lower staff accommodation building. We are in receipt of a plan showing the location of these sub-soil drains as drawn by Michael Samms & Associates Pty Ltd (Drawing Number 99-629/09, sheet No. C-01, Issue B dated 1 February 2002). The sub-soil drain locations are shown on the attached Figure 3. We understand that this drainage was installed in mid 2002, although Jeffery and Katauskas were not involved in the installation or design of the subsoil drains. However from the drawings we understand that the sub-soil drains essentially comprise a 100mm diameter geofabric covered Class 1000 slotted pipe within a slotted 150mm diameter UPVC outer casing.

3 STUDY METHODOLOGY

The methodology for this additional geotechnical assessment has included the following stages;

- Data gathering from groundwater and inclinometer monitoring.
- Review of rainfall records for the period of the assessment and comparison with historical rainfall records as a basis for evaluation of return periods for specific events during the monitoring period.

- Comparison of data obtained from current monitoring to previous hydrogeological models and assumptions, including groundwater level responses to rainfall.
- Site specific slope stability analysis to evaluate factors of safety for recorded groundwater results, including sensitivity to soil shear strength parameters.
- Review of likely geotechnical processes and potential modes of slope movement.
- Assessment of results and review of quantitative risk analysis and in particular the probability of landsliding.

Components of these stages are outlined in more detail below.

3.1 Groundwater and Inclinometer Monitoring

Groundwater and inclinometer monitoring was carried out along three section lines (Transect 1, Transect 2 and Transect 3). The transect lines and monitoring locations are shown on the attached Figure 2. For specific details reference should be made to the WRL TR 2007-32. Some of the specific details are discussed below.

Groundwater monitoring was carried out from piezometers comprising 50mm Class 18 PVC with a 0.75m screened interval at the base of the borehole. Each piezometer was fitted with an Odyssey water level logger set to automatically record water level readings at 30 minute intervals. In addition YAM3A was fitted with a DIVER and BARO-DIVER logger (Refer to Table 4 in WRL TR 2007-32). Odyssey water level loggers were also installed in previous monitoring wells MS1, MS5 and MS9.

Inclinometer casing was installed in separate boreholes immediately adjacent to 6 of the 8 new piezometer locations; being Locations 1A, 1C, 2A, 2C, 3A and 3C. The inclinometer casing is grouted into the underlying bedrock. Measurements are made by inserting an inclinometer probe down the casing to the base and then reading the tilt of the casing at 0.5m intervals as the probe is retracted from the casing. The tilt readings are compared to an initial reference reading taken soon after installation in order to produce a plot of horizontal movement against depth.

3.2 Rainfall Data

The daily rainfall data was obtained from a pluviometer located at Yamba Pilot Station, on Pilot Hill adjacent to the study area. A statistical analysis of the rainfall records from the monitoring period (May 2005 to June 2007) was carried out to assess the rainfall return periods (including antecedent rainfall) in relation to historical rainfall records and return periods. Only one reasonably significant rainfall event occurred during the monitoring period. This event occurred on 30 June 2005 where a daily rainfall of 250.4mm occurred. This rainfall event has formed the basis of most of the groundwater analysis and development of hydrogeological models.

3.3 Hydrogeological Models

The previous hydrogeological models presented in the Jeffery and Katauskas report of October 2000 were based on simplified groundwater flow models and permeability assumptions. The current monitoring has allowed direct measurement of the groundwater levels and their response to rainfall events. Therefore more direct hydrogeological models for the study area based on the monitoring results could be established.

3.4 Geotechnical Slope Stability Analysis

Using the subsurface data obtained from the boreholes used to install piezometers and the monitored groundwater data, subsurface geotechnical models at each of the three transect lines were set-up. Slope stability analysis was carried out on each transect to calculate a Factor of Safety (FOS). The geotechnical models included groundwater levels recorded during the monitoring period and some sensitivity analysis on the soil shear strength parameters and groundwater levels for Transect 1. Further details are discussed in Section 5 below.

3.5 Geotechnical Processes

Using the groundwater monitoring results, the slope stability results and the results from the inclinometer monitoring it is possible to assess the likely geotechnical processes and modes of potential failure within the slope. This has been assessed and is discussed further in Section 6 of this report.

3.6 Quantitative Risk Assessment

The previous Jeffery and Katauskas report (October 2000) included a historical data search in relation to the history of slope instability. It was not within the scope of this additional assessment to carry out further historical searches. Reference should be made to our previous report for details. That previous search, combined with the results of the recent monitoring and analysis have been used to refine our assessment of the probability of landsliding in the study area.

4 RESULTS OF MONITORING

4.1 Groundwater Monitoring and Rainfall Analysis

The Water Research Laboratory (WRL) has produced plots of the groundwater level vs rainfall for each of the monitoring points. These have been presented with this report as Figures 4 to 7. It is important to note that where no groundwater data has been recorded we understand that generally the groundwater level was below the base of the data logger, although there also appears to have been some lost data for location 3A in September 2006. These plots have been produced from the raw groundwater data and are discussed in more detail below.

4.1.1 Groundwater Monitoring

Transects 1, 2 and 3

WRL have produced Tables 4 to 9 in their TR 2007-32 which summarise the groundwater monitoring. It is not the intent of this report to replicate their summary of results, but to provide broad comment on the results in regard to slope stability. A comparison between the conceptual groundwater models (prepared in the Jeffery and Katauskas October 2000 report) with groundwater monitoring results during the June 2005 rainfall event (which produced the highest daily rainfall over the monitoring period) was completed. Considering the various unknowns and variables the comparison indicated that the conceptual model was very close (well within the expected range for this type of analysis) to the actual conditions encountered during the June 2005 rainfall event. Of significant importance to stability the following was recorded for the June 2005 event;

- Groundwater levels rose by an average of 286mm for 100mm of rainfall. This equated to rises in the order of 0.75m to 1.74m above 'steady state' groundwater conditions (or groundwater conditions prior to the rainfall event). This confirms that significant groundwater rises can occur after even single day rainfall events

- The largest groundwater rises occurred at the toe of the slope in Transect 1 and 2, while it occurred midslope in Transect 3. We expect that the lower rise at the toe of the slope in Transect 3 was due to the groundwater virtually reaching the surface during the rainfall event and thus further rises were not possible as seepage was exiting the slope. This confirms that the toe of the slope will virtually become saturated during rainfall events.

- The time lag between rainfall and groundwater level rise at the crest of the slope is about 3 days. At the toe of the slope where the sand profile is significantly shallower, the time lag between rainfall and groundwater level rise is between 0.7 days and 1.4 days. This also confirms that a single rainfall event can cause a relatively quick saturation of the toe of the slope.

- Based on the results of piezometer 1C the total time to drain the slope after the rainfall event is 96 days (or an average of about 16.3mm per day). However there is a higher rate of groundwater decline in the early days after the event (77.2mm/day up to day 11 after the event). This early decline is about half of the total rise (i.e. for a total decline in groundwater level of 1.6m, a groundwater level decline of 0.85m occurs in the first 11 days after the rainfall event). This early 'more rapid' decline is followed by a slower decline of only 6.3mm/day. This trend is reasonably typical for each of the piezometers at the toe or middle portions of the slope and confirms groundwater levels remain elevated (even if only at half the initial peak) for some time after the rainfall event.

- At the crest of the slope the groundwater decline for piezometer 1A and 2A show a more uniform rate of groundwater decline with time (in the order of 10mm/day, based on a decline of about 0.7m in two months). 3A also shows a more uniform rate of groundwater decline after the rainfall event (in the order of 30mm/day, based on about a 1m rise taking about 1 month to decline to original levels). This confirms that at the crest of the slope, ground water levels remain elevated for some time (2 to 3 months) after the rainfall event. Therefore more continuos rainfall (antecedent rainfall) will be the most critical element in producing groundwater levels which may cause instability of the slopes. At this stage 30 day or greater antecedent rainfall events with a higher return period than about 3 years have not yet occurred in the monitoring period (refer to Section 4.1.2 below). Therefore we recommend rainfall and groundwater level monitoring should continue at this stage so that results for higher antecedent rainfalls can be assessed in regard to stability.

Yamba MSA Transect

This transect included monitoring wells MSA1, MSA5 and MSA9, which are located on the mid to lower northern slopes of the Pacific Hotel. These monitoring wells were installed prior to construction of the sub-soil drains in mid 2002. The MSA monitoring wells were constructed with the screen over the entire length. The following Tables 4.1.1(a) and 4.1.1(b) summarise the previous monitoring results (From about October 2000 to April 2001, or prior to installation of the sub soil drains), and the more recent monitoring (June 2005 to July 2007, after installation of the sub-soil drains). We also note that in March 2001 there was a relatively similar rainfall pattern to that which occurred in June 2005 and so comparison of the two events with and without the sub-soil drains appears reasonable.

From the tables below it is possible to look at the affect that the sub-soil drains are having on the groundwater levels and groundwater declines during periods of rainfall. Since these MSA monitoring wells are fully screened the groundwater responses may be influenced by the larger screen length compared to the more recent installations.

MSA1 is located upslope and about 13m from the subsoil drains as shown on Figure 2. Comparison of the results indicates that there has been negligible affect on groundwater levels or rates of groundwater decline from the installation of the subsoil drains.

MSA5 is located about 3.5m from the sub-soil drains and close to the Pacific Hotel as shown on Figure 3. While there was not much data obtained from MSA5 during the current monitoring period; it is likely that this is due to the groundwater being below the base of the data logger. Nevertheless, comparison of the results indicated a reduction in groundwater level rise during the June 2005 event (after the sub-soil drains were installed) compared to the March 2001 event (before the sub-soil drains were installed).

Table 4.1.1(a) Summary of Groundwater Results Prior to Installation of Sub-Soil Drains

Monitoring Well Location	Groundwater Levels (GWL) Lowest RL	Highest RL	Associated Responses
MSA1	22.1m	23.2m	• GWL rise of about 760mm for 309mm rain during 7 days. • GWL decline overall about 430mm drop over 18 day period (24mm/day) • GWL decline peak about 300mm drop over 4 day period (75mm/day)
	Range = 1.1m		
MSA5	17.2m	18.2m	• GWL rise of about 960mm for 309mm rain during 7 days. • GWL decline overall about 846mm drop over 17 day period (50mm/day) • GWL decline peak about 336mm drop over 4 day period (84mm/day)
	Range = 1.0m		
MSA9	16.13m	17.48m	• GWL rise of about 1100mm for 309mm rain during 7 days. • GWL decline overall about 670mm drop over 20 day period (33.5mm/day) • GWL decline initial peak about 450mm drop over 5 day period (90mm/day)
	Range = 1.35m		

NOTE: The above table is based on an event comprising heavy rainfall around 9 March 2001 which gave a maximum single day rainfall of about 190mm; with 309mm of rain over a 7 day period.

Table 4.1.1(b) Summary of Groundwater Results After Installation of Sub-Soil Drains

Monitoring Well Location	Groundwater Levels (GWL) Lowest RL Highest RL	Associated Responses
MSA1	22.1m 23.16m Range = 1.1m The lowest RL of 22.1m appears to be the base of the data logger.	• GWL rise of about 860mm for 376mm rain during 5 days. • GWL decline overall about 460mm drop over 18 day period (26mm/day) • GWL decline peak about 260mm drop over 4 day period (65mm/day)
MSA5	17.2m 17.4m Range = 0.2m The lowest RL of 17.2m level appears to be the base of the data logger	• GWL rise of about 200mm for 376mm rain during 5 days. • GWL decline overall about 100mm drop over 9 day period (11mm/day) • Appears water dropped below base of logger or error with logger.
MSA9	13.5m 15.4m Range = 1.9m The lowest RL of13.5m lowest level appears to be base of the data logger.	• GWL rise of about 700mm for 376mm rain during 5 days. • GWL decline overall about 400mm drop over 20 day period (20mm/day) • GWL decline initial peak about 200mm drop over 5 day period (40mm/day)

NOTE: The above table is based on an event comprising heavy rainfall around 30 June 2005 which gave a maximum single day rainfall of about 250mm; with 376mm of rain over a 5 day period.

MSA9 appears to be located almost directly over the sub soil drains as shown on Figure 2. The highest level that the groundwater reached during the current monitoring period is about 2m lower than prior to the installation of the sub-soil drains. There was also slightly less of a rise in groundwater (700mm rather than 1100mm) during the June 2005 event compared to the March 2001 event. Therefore it appears that the sub-soil drains have lowered the groundwater at the sub-soil drain location, although groundwater rises during rainfall events are still occurring even close to the sub soil drains.

At this stage we have no indication of the affect that the subsoil drains have on the groundwater levels at the toe of the slopes and this would need to be assessed further if stabilisation using drainage is proposed.

4.1.2 Rainfall Analysis

We have carried out some rainfall analysis to look at the rainfall records over the period of monitoring. We have looked at daily rainfall and the rolling total rainfall over periods of 2 days to 90 days throughout the monitoring period. These rolling totals represent the antecedent rainfall over the preceding period up to and including the given date. Table B1 (3 pages) in Appendix B summarises the top 25 ranked events. We have then plotted the highest ranking event for each case on a standard Gumbel plot. The Gumbel plot used was prepared during our previous geotechnical assessment in October 2000. That previous plot included rainfall records from 1877 to 1999. The plot has not been updated to account for the last seven years of rainfall, however given the relatively low rainfall in that period we do not believe that there would be any significant change to the plot. The plots are attached as Figures B2 to B4. The following Table 4.1.2 summarises the highest ranking antecedent rainfalls and the return periods.

As can be seen from the table below the antecedent rainfalls are predominantly governed by the one significant event at the end of June 2005. It is also clear from the groundwater vs rainfall plots in Figures 4 to 7 and from the table below that there has not been much rain either preceding or following the rainfall event in June 2005 or at any other times during the monitoring period. This single day event had a return period of 45 years (which means simply that such an event is only likely to occur on average once every 45 years). This single day event was the only single day event which exceeded a 10 year return period over the duration of the current monitoring. Therefore the groundwater monitoring results are relying on essentially a significant single day rainfall event and not on any significant antecedent rainfalls.

Table 4.1.2 Summary of Highest Ranking Rainfall Events and Return Periods

Period of Antecedent Rainfall	Total Rainfall (mm)	Date of Event * (See Note Below)	Approximate Return Period
1 Day Rainfall	250.4	30/06/2005	45 years
2 Day Rainfall	323.4	1/07/2005	30 years
5 Day Rainfall	376.3	1/07/2005	20 years
10 Day Rainfall	385.7	5/07/2005	10 years
15 Day Rainfall	388.4	2/07/2005	6 years
30 Day rainfall	405.5	24/07/2005	3 years
45 Day Rainfall	426.1	1/07/2005	1 to 2 years
90 Day Rainfall	600.2	30/07/2005	1 to 2 years

*Note the date of the event represents the end of the period of antecedent rainfall

The groundwater level vs rainfall plots, show that it takes some time for the slope to 'drain' (i.e. for groundwater levels to reduce back to levels prior to any rainfall event). Refer to section 4.1.1 above

Therefore groundwater monitoring should continue so that the Hydrogeological models can be modified to account for any future groundwater responses to higher antecedent rainfall events.

4.2 Inclinometer Monitoring

Inclinometer monitoring was carried out during the period of 29 April 2005 (initial baseline set-up reading) to 8 March 2007. All inclinometers with the exception of Location 2C indicated little or no significant movement. However Location 2C showed 5mm of movement with the plane of movement at a depth of about 3m.

Sandy clays exist at 3m depth in Borehole 2C, therefore the movement is occurring within the sandy clays.

The mode of movement is consistent with the stability results for Transect 2, which shows that the lowest factor of safety occurs for slip circles at the toe of the slope when the higher groundwater levels exist during rainfall periods. The movement in Inclinometer 2C tends to suggest a slow moving (Creep) type movement occurring at the toe of the slope.

5 GEOTECHNICAL STABILITY ANALYSIS

5.1 Surface Conditions and Geometry

Surface survey measurements have been taken for each of the transects. The geotechnical surface conditions have been based on these survey measurements, although we have also used our own observations to refine the surface model where necessary.

Transect 1 is located immediately to the south of the Pacific Hotel and is characterised by relatively steep upper slopes in the order of 30° to 35°. Midslope there is an unformed accessway know as Marine Parade which traverses the slope. Below Marine Parade slopes range from 20° to 35° down to the beach area. Along Transect 1 the slopes are typically only 20° to 25°, however just to the north the foreshore slopes are 30° to 35° with a foreshore cliff line developing to the north. The Pacific Hotel is located within the upper steep slopes.

Transect 2 is located on the northern side of No.14 Pilot Street and extends down to the beach area. This section also has relatively steep upper slopes in the order of 30° which flatten out to about 15° to 20° above Marine Parade. Below Marine Parade the foreshore slopes are in the order of 30° to 35° with a rocky cliff line forming the toe of the slope. Residential dwellings are located within the upper steep slopes and also within the lower flatter regions.

Transect 3 is located within a public pathway to the north of No. 8 Pilot Street. This transect has upper slopes flatter than the other two transects and only in the order of 15° to 20°. The foreshore slopes are similar to other areas with steep slopes of about 30° down to a rocky cliff line at the toe. Residential dwellings are located within the upper slopes.

5.2 Subsurface Conditions

We have used the borehole data obtained during installation of piezometers and inclinometers to form a geotechnical subsurface model at each of the transects. The borehole logs prepared by WRL have been included with this report in Appendix C. These boreholes have been complimented by reference to previous investigation data around the Pacific Hotel to enhance the subsurface models where possible. The subsurface models are shown on in Appendix A as Figures A1 (Transect 1), A20 (Transect 2) and A28 (Transect 3). Interpolation and judgement of likely subsurface conditions has been made between the known conditions at the borehole locations.

Typically the profiles comprise an upper sequence of sands and silty sands generally of medium dense relative density (although the upper sands can also be loose, particularly at the crest of the steeper foreshore slopes). Over the upper slopes, sandy clays are encountered below the sands at depths in the order of 5m to 11m, with the greater depths at the southern end closest to the Pacific Hotel (Transect 1) and the shallowest depth at the northern end (Transect 3). At transect 1 and 3 there appears to be little or no sandy clay below the sands from about midslope down to the foreshore cliff line. However Transect 2 appears to contain a reasonably uniform layer of sandy clay from the upper slopes down to at least the crest of the steeper foreshore slopes. The sandy clays appear to be of generally very stiff strength.

Weathered sandstone bedrock was encountered below the sandy clays and it was encountered at depths typically ranging from 8m to 16m below the crest of the upper slopes and between 1.5m and 4m below the crest of the foreshore slopes.

5.2.1 Material Properties

Based on the limited insitu testing carried out during the piezometer installations, we have adopted the following soil shear strength parameters for our stability analysis. In our opinion the values below are 'realistic' for the subsurface conditions encountered. It is possible that the soils may exhibit higher or lower shear strength properties and therefore we have also carried out some limited sensitivity analysis by varying the soil shear strength parameters.

Sands and Silty Sands

An effective cohesion (c') = 0kPa, and effective friction angle $\varnothing' = 35^{\circ}$ for the sands. At this stage we do not believe that there is any real justification for higher shear strength parameters through the sands, although in the upper partially saturated zone, the sands may have some slight cohesion and any denser layers may have a slightly higher effective friction angle.

An effective cohesion (c') = 2kPa, and an effective friction angle $\varnothing' = 35^{\circ}$ for the silty sands. There may be some slight cohesion due to cemented layers within the silty sands which have been identified by others to occur in pockets throughout the profile, as such the silty sands have been given some slight cohesion. Higher effective friction may occur in any denser or cemented layers.

For an assessment of the sensitivity of the soil shear strength parameters we have also run a stability model on Transect 1 using what we consider to be upper bound parameters for the sands ($c' = 2kPa$ and $\varnothing' = 37^o$), and silty sands ($c' = 5kPa$ and $\varnothing' = 37^o$).

Sandy Clays

An effective cohesion $c' = 10kPa$ and effective friction angle $\varnothing' = 25^o$. Within the clays we do not believe that higher shear strength parameters can be justified without specific strength testing.

Bedrock

The underlying bedrock is relatively strong in comparison with the overlying soils therefore the analysis has assumed that failure surfaces would not pass through the bedrock.

5.2.2 Groundwater Levels

The stability analysis has been undertaken using groundwater data obtained from the period of monitoring. Two cases have been analysed a high water case and a low water case.

The low groundwater or 'steady state' groundwater conditions are those levels where groundwater levels tend to verge toward prior to and after rainfall events. The groundwater level in this case is typically either at the surface of the sandy clays or weathered bedrock, although at Transect 1 the groundwater level at the crest of the upper slopes is about 3m above the top of the sandy clays.

The high groundwater case is the highest groundwater level recorded in the monitoring period, which has been taken from the groundwater response during the most significant rainfall event at the end of June 2005, where 250mm of rain occurred in one day.

We have also analysed the Transect 1 model using inferred groundwater levels taken from the monitoring wells YAMMSA1, YAMMSA5 and YAMMSA9. This has been carried out to assess the variation in FOS due to the installation of the sub-soil drains. We have considered both low groundwater levels and high groundwater levels (as recorded during the June 2005 rainfall event).

To assess the possible sensitivity of the groundwater levels we have also analysed Transect 1 using groundwater levels uniformly 0.5m higher than occurred for the June 2005 rainfall event.

5.3 Methods of Stability Analysis

Stability analysis has been carried out using the computer program SLOPE/W considering circular failures. This program uses a routine analytical procedure whereby failure circles of different radii and location are considered and analysed for stability. Each circle is divided into vertical slices and the stability of each of these slices summated to provide a Factor of safety (FOS) for the overall failure arc.

The computer stability analysis has been carried out using the automatic generation of failure circles as incorporated within the program. A number of failure circles are evaluated for each centre within a defined grid of slip circle centres. At each centre a number of slip circle radii are considered. The result of the analysis of each circle is expressed as a FOS, which is the ratio of the resisting moments (or forces times the radius of the slip circle) divided by the disturbing moments (or forces times the eccentricity from the slip circle centre). For a FOS of 1.0, instability is incipient. FOS values of less than 1.0 imply that failures should have occurred, whilst for FOS values greater than 1.0, failure should not occur. As the FOS is dependent upon the geometry of the surface and subsurface layers, shear strength of the layers and the groundwater levels, then clearly the actual FOS may be different to the computed value, depending on the accuracy of these assumptions.

5.4 Results of Stability Analysis

We note that although the stability analysis has assumed a circular failure that in nature this is not always the case. However the circular failure is a reasonable approximation for many failures. Similarly, the calculated FOS is also indicative of the overall stability for failure surfaces which may be non-circular but can be reasonably approximated by failure circles of a similar size and shape. Thus the precise failure surface may not match the circular analysis. However, the broad trends for location of the areas of lowest stability, and areas of instability, may be derived from the circular analysis.

5.4.1 Overview

Overall the stability analysis of the three subsurface models has shown that the slopes have low FOS, particularly for the higher groundwater levels and the section close to the Pacific Hotel (Transect 1), where it could be said that the stability is marginal with FOS close to 1.0. We note that the FOS values calculated from the computer stability analysis have been rounded off to 1 decimal place to account for the generalised nature of the subsurface models.

A summary table of the FOS for specific circles is presented in Table A1 of Appendix A. For a graphic description of the specific circles, reference should be made to Figures A2 to A34 as indicated within Table A1. Results for the critical circles for each subsurface model are discussed separately below.

It can be seen from the results outlined below that the FOS values are generally less than the usually accepted values which are normally 1.5 for a 'reasonable design case'. Values as low as 1.25 may be tolerated for transient short term conditions. We note that some of the analyses, particularly for Transect 1 show that under likely groundwater level increases, the FOS would be less than 1.25, and this would be regarded as unacceptable.

The critical issue in terms of stability and determination of acceptable or unacceptable FOS will be to plot the FOS verse annual return period of groundwater levels. At this stage we do not have sufficient rainfall and groundwater monitoring data to prepare such a plot for any of the higher antecedent events. It may be possible to construct a reasonable groundwater model to predict the likely groundwater changes during these higher antecedent events and to input the results into a slip circle stability analysis to obtain FOS under these predicted conditions. It is currently not within the scope of this report to carry out such a detailed groundwater model, but we consider that such a model would be beneficial to future revision of probability assessments and also stabilisation/treatments works where an 'acceptable' FOS will need to be determined.

5.4.2 Subsurface Model – Transect 1 (Figures A1 to A20)

The slip circle results for Transect 1 have shown there are essentially two specific locations on the slope where failures are most likely to occur, and there are three specific failure modes.

- The first area is the toe of the slope, where the foreshore slopes intersect the beach level. The minimum FOS for this toe has been shown to be a moderate slip circle which has a FOS of 1.4 (High Groundwater Case - Refer to Figure A7). This type of potential failure is particularly evident below the Pacific Hotel (i.e. just to the north of Transect 1, where the foreshore slopes are steeper and FOS values would be expected to be lower and more like Transect 2 toe slopes).

- The second involves potential failures within the upper slopes. There are two types of potential failures that occur within these upper slopes; small shallow surface failures, and deep seated failures. The shallow surface failures (Figure A2) have FOS in the order of 1.1 and are generally evident as surface creep of the sandy slopes leading to movements in plumbing, fence lines etc, as seen on site. The deep seated failures (similar to that which occurred at the front of the Pacific Hotel in the 1950's) is typical of the deep seated failures (Figure A3 and A6). The deep seated failures also have FOS of 1.1 for the high groundwater case.

- To indicate the critical nature of the groundwater level in the slope at Transect 1 we have also carried out an analysis assuming the groundwater level rises uniformly by only an additional 0.5m from the groundwater levels measured during the June 2005 rainfall event. Considering the low antecedent rainfalls experienced over the monitoring period, such additional rises are considered likely. It can be seen from Figure A12 that the FOS for a deep seated failure reduces to 1.0.

- We have also looked at the FOS for the case where groundwater levels have been lowered by sub-soil drainage below the Pacific Hotel (Figures A13 to A20). Based on the groundwater levels from YAMMSA1, YAMMSA5 and YAMMSA9, an inferred groundwater profile for a high groundwater case (taken from the June 2005 rainfall event) has been adopted for the Transect 1 model. For a deep seated failure the FOS was 1.3 (Figure A19), which could be considered as only just tolerable under short term transient conditions.

- As discussed in Section 5.2.1 above, we have also carried out some limited sensitivity analysis for the Transect 1 model assuming some upper bound soil shear strength parameters. For the critical deep seated slip circle failure (Figure A9) the FOS increased to 1.2 (from 1.1). This indicates that the model is not overly sensitive to an increase in shear strength parameters. Such a FOS would still be considered unacceptable. The higher soil shear strength parameters could explain why there have been no recent large failures for known past rainfall events.

5.4.3 Subsurface Model – Transect 2 (Figures A21 to A28)

Transect 2 has also shown essentially two specific areas of failure. The upper slopes and the toe (or foreshore slopes)

- Slip circle failures at the toe of the slope have relatively low FOS values of 1.2 (Figure A26) for the high groundwater level case. The model assumes a uniform groundwater level surface from Piezometer 2C down to the crest of the cliff face. In reality this slope may even be completely saturated during rainfall events, which would lead to lower FOS values. Such slope failures are evident in the foreshore slopes above the cliff faces near to Transect 2. The inclinometer 2C shows some movement of the slope and this could also indicate the likelihood that the lower portion of the slope becomes saturated during rainfall leading to creep or slow movements of the foreshore slopes.

- Critical slip surfaces within the upper slopes have FOS values of 1.2 for shallow surface type failures (Figures A21 and A25) indicative of surficial creep type movements leading to movements in fence lines, cracking of small retaining walls etc. Deeper slip circles in the upper slopes tend to have an incremental increase in FOS with depth (Figures A23 and A27)

5.4.4 Subsurface Model – Transect 3 (Figures A29 to A35)

For Transect 3, the main area of failure is the foreshore slopes and the toe of the upper slopes.

- The critical slip circles for the foreshore slopes have FOS values of about 1.2 for the low water case and less than 1.0, for the high water case (Figures A29 and A32). In this area there is evidence of these types of failures in the foreshore slopes above the cliff line.

- At the toe of the upper slopes, FOS values for moderate sized slip circles during high groundwater levels are about 1.6 (Figure A33). We have not observed any such failures in these areas. Higher FOS values occur for larger deep seated slip circles encompassing the upper portions of the upper slope.

6 GEOTECHNICAL PROCESSES

The results of the groundwater monitoring, the slope stability analyses and the inclinometer measurements, indicates that the most likely geotechnical processes affecting stability and failure modes of the slopes in the study area are as follows;

Transect 1

High antecedent rainfall, which causes an increase in groundwater level through the upper, mid and lower portions of the upper slopes. This results in the potential for deep seated failures. Immediately to the north of Transect 1, saturation and failure or movement of the foreshore slopes may be a precursor to deep seated failures in the upper slopes.

Transect 2 and 3

High daily and antecedent rainfall causes saturation of the foreshore slopes and produces low FOS for this area. This is particularly evident where the foreshore slopes are steepest and intersect the rock cliff face above the beach. Movement of the toe of the slope can instigate movements within the upper slopes, such as by regression of smaller failures or loss of passive support at the toe for larger failures. Deep seated slope instability could occur (prior to failure or movements of the foreshore slopes) where the upper slopes are steeper, however these appear to be less likely. Regression of the cliff faces themselves may also instigate slope instability upslope. While cliff face regression is on average per year quite slow, in reality it occurs in larger sections, infrequently, which could remove some of the toe of the foreshore slopes. Reference can also be made to WRL TR 2007/32 for further discussion on cliff face regression.

7 RISK ANALYSIS

Our previous geotechnical assessment (October 2000 Report) carried out a detailed risk assessment including the following;

- Rainfall Analysis, and
- Probability Assessment.

These items were used in combination with an assessment of the consequences of landsliding to calculate the risk to life and to assign landslide risk zones. The rainfall analysis is discussed in Section 4.1.2 above. Some further summarised discussion on probability assessment is included below.

7.1 Probability Assessment

The probability assessment included;

- A historical search for known landslides.
- Evaluation of the actual rainfall and antecedent rainfall for known landslides.
- Production of a summary of return periods of actual rainfall and antecedent rainfall for landslide events.

Summary tables of the probability assessment which were produced as part of our previous October 2000 geotechnical assessment have been provided in Appendix D of this report for reference.

The probability assessment included all known landslides and events as could reasonably be determined from the historical search, with the earliest known event occurring in May 1938. Further details of the historical search and probability assessment are contained in our previous geotechnical assessment report.

We are not aware of any further landslide events within the study area from October 2000 to the end of the current monitoring period. Therefore considering the similarity between the current hydrogeological models and previous assumptions we consider that the probabilities previously adopted are still reasonable. However the probability estimates calculated in our previous geotechnical assessment were based on landslides occurring each time the 'trigger' rainfall or antecedent rainfall occurred. Further review of the probability estimates since our October 200 report indicates that only about 50% to 70% of the time that a certain 'trigger' rainfall or antecedent rainfall was reached did a landslide event occur. Therefore the previous probability estimates based on the historical data have been reduced by 50% in this updated risk assessment.

Indicative Annual Probabilities

From the probability assessment, we consider that the following may be regarded as indicative annual probabilities.

- For earthslides and scour at the toe of slopes (foreshore slopes)

  Trigger events are 1 in 10 years to 1 in 125 years

  (i.e. previously assessed probability of 10^{-1}per annum to $8x10^{-3}$per annum)

However considering that landslides only occur about 50% of the time a 'trigger' level is reached, then this equates to a probability of $5x10^{-2}$ per annum to $4x10^{-3}$ per annum.

- For earthslides encompassing the steeper hillside slopes (upper slopes)

  Trigger events are 1 in 10 years to 1 in 100 years

  (i.e. previously assessed probability of 10^{-1} per annum to 10^{-2} per annum)

However considering that landslides only occur about 50% of the time a 'trigger' level is reached, this equates to a probability of $5x10^{-2}$ per annum to $5x10^{-3}$ per annum.

Using the above revised probability estimates we have revised the risk table from our previous report and attached this table as Table F in Appendix D.

7.2 Risk Analysis Results

The general principle in risk analysis is that the RISK is a product of the PROBABILITY that an event will occur and the CONSEQUENCES if the event does occur. In this instance where we are assessing the risk to life, the Consequences are loss of life, however the Probability can be further broken down to some partial probabilities such as;

- If the event occurs will it impact an element (e.g. a structure or person). This is called a Spatial Probability.
- If the event occurs and the element is impacted what is the probability that people will be within or using the element at the time of the event occurring. This is called the Temporal Probability.
- If people are impacted by the event what is the probability that there will be loss of life. This is called the Vulnerability.

These partial probabilities have been taken into consideration in our risk analysis shown in Table F.

The following risk estimates have been considered in relation to the suggested criteria given in AGS (2007) Risk Management Guidelines which are;-

For an existing slope: **Tolerable Risk of 10^{-4}** for loss of life for person most at risk.
Acceptable Risk of 10^{-5} for loss of life for person most at risk.

It will be up to the owners and regulators to decide whether these values are appropriate and the conclusions regarding the risk estimates reasonable. The risk estimates should also be considered in the light of the FOS values for the models analysed.

The highest risk values are associated with Landslide Risk Zone 1a (LRZ1a) (encompassing the Pacific Hotel), where slopes are steepest, the history of movement most evident, occupancy is expected to be high and the probability of a larger scale failure affecting the Pacific Hotel (Spatial probability) is higher. The specific results for the risk assessment are shown in Table F of Appendix D. However in summary the results obtained for LRZ1a are:-

For a) Very Slow to Slow Movements 7.5×10^{-5} to 1.5×10^{-5}, say 5×10^{-5}
TOLERABLE

For b) Rapid to Very Rapid Movements 2×10^{-3} to 4×10^{-4}, say 10^{-3}
UNACCEPTABLE

LRZ1b includes residential dwellings in the area close to the Pacific Hotel. This zone also includes the area where the sub-soil drains have been installed. It is possible that the probability of landsliding, particularly for deep seated slides may be able to be reduced in LRZ1b on the account of the existing sub-soil drains. However at this stage although there is some evidence that the sub-soil drains have lowered the groundwater, there is insufficient information upslope, downslope and close to the drains to provide any assurance of the effectiveness of the drains to maintain lower groundwater levels and significantly reduce the risk of landsliding. In addition the computer based stability analysis has still indicated FOS values of about 1.3 for the deep seated failure and a high groundwater level. Such a FOS indicates possibly just tolerable conditions depending on future analysis of antecedent rainfall effects on groundwater levels. Therefore the results of the risk assessment are: -

For a) Very Slow to Slow Movements $2x10^{-5}$ to $2x10^{-6}$, say 10^{-5}
ACCEPTABLE (Just)

For b) Rapid to Very Rapid Movements $7.2x10^{-4}$ to $7.2x10^{-5}$, say $4x10^{-4}$
UNACCEPTABLE

In order to consider any further reduction in the probability of landsliding due to the sub-soil drains, further groundwater monitoring upslope and downslope would be required, as well as an assessment of the extent (or distance) of groundwater lowering away from the drains.

The other LRZ (being LRZ1c, LRZ2 and LRZ3) are all considered to have acceptable risks to life with risks less than or about equal to 10^{-5}.

The attached Figure 8 shows the inferred boundaries between the LRZ's. We do not believe that the data obtained from the current groundwater monitoring allows any adjustment to these zones from that provided in our previous geotechnical assessment in October 2000.

8 ALERT LEVELS

From the results of our previous geotechnical assessment (October 2000) an interim management strategy was put into place to try to manage the risk. This management strategy was aimed at identifying possible rainfall conditions that may trigger a landslide event. It was considered that conditions that may give rise to an emergency are any of:

a) A period of prolonged high rainfall, say over 30 days to 90 days

b) A period of high daily rainfall after previous wet periods

c) High intensity rainfall over short periods of say 1 day or less.

From examination of the data, emergency rainfall warning levels were set up. Two warning levels were assigned; an Orange level which was based on a 1 in 3 year rainfall, and a Red level which is based on a 1 in 10 year rainfall. The relevant rainfall warning levels are given in the table below.

Relevant Rainfall Warning levels

Antecedent Rainfall Period (days)	Orange Level (mm)	Red Level (mm)
1	180	200
2	200	280
5	215	325
8	250	370
15	310	425
30	425	560
45	500	675
60	600	800
90	740	955

From our recent assessment of the groundwater monitoring, inclinometer monitoring, slip circle analyses and revised risk assessment, we consider that the above emergency levels and the subsequent management implications should remain in place until more permanent stabilisation measures are adopted.

9 STABILISATION

Following our review of the geotechnical monitoring and assessment presented in this report, we are of the opinion that the various treatment/stabilisation options presented in our previous report (October 2000) should be investigated in more detail with a view to implementation as soon as possible.

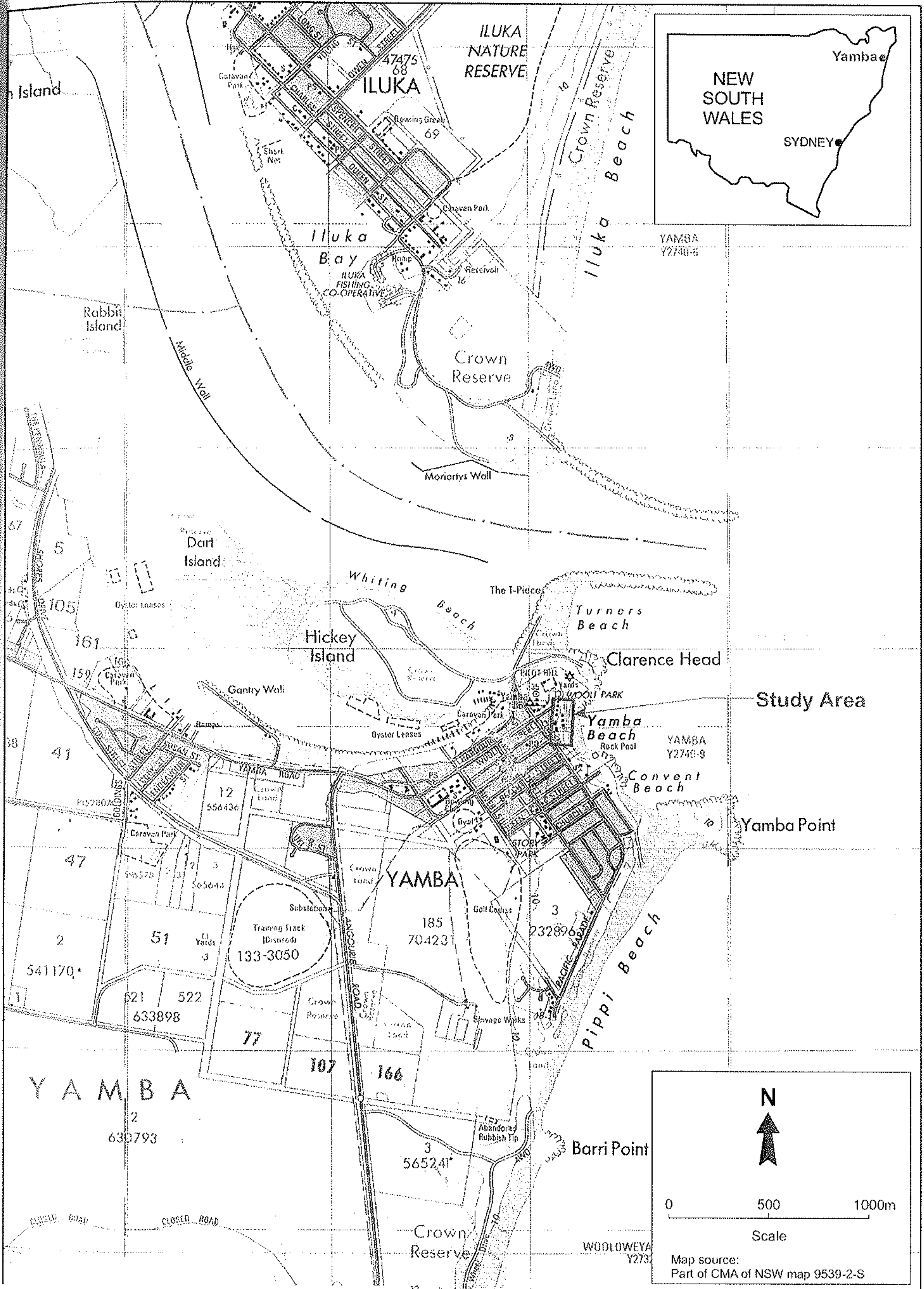
Continuous groundwater monitoring using the existing data loggers should also continue so that our knowledge base on groundwater responses with rainfall events can increase and groundwater models can continue to be refined. This is particularly important so that the groundwater responses to any higher antecedent rainfall events can be observed and modelled.

We also recommend continued inclinometer monitoring, although we believe that the frequency could be reduced to yearly or even longer, unless significant rainfall events occur and/or movements of the slope are observed, in which case the inclinometers should be read as soon as possible.

Should you have any queries regarding this report, please do not hesitate to contact the undersigned.

LJ Speechley
Senior Associate

for BF Walker
Principal
For and on behalf of
JEFFERY AND KATAUSKAS PTY LTD.

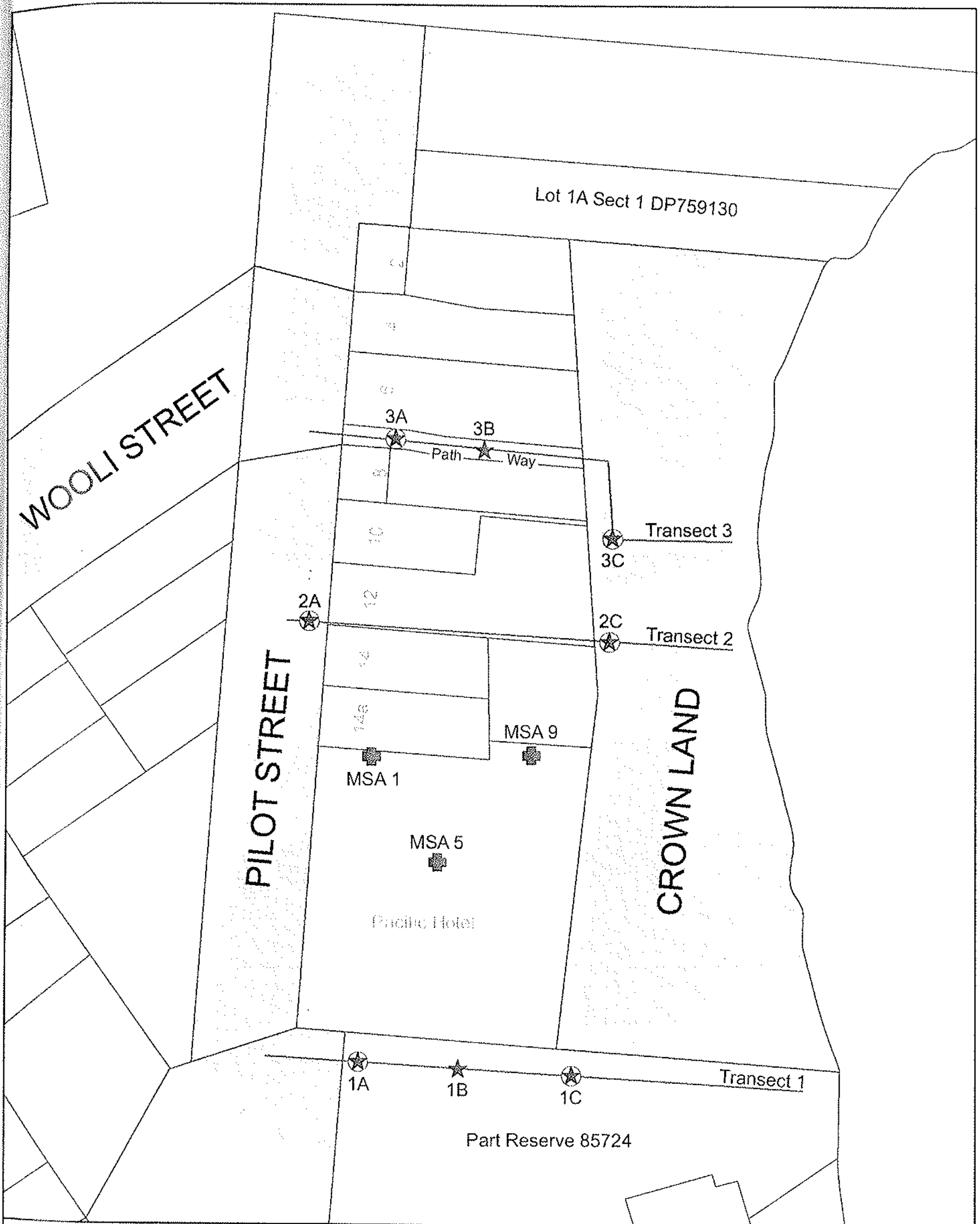


Jeffery and Katauskas Pty Ltd

Report No. 19314WL2 Figure No. 1

LOCATION MAP

Figure 1



Map courtesy of Clarence Valley Council

LEGEND:

- Inclinometer
- Piezometer
- Existing monitoring site

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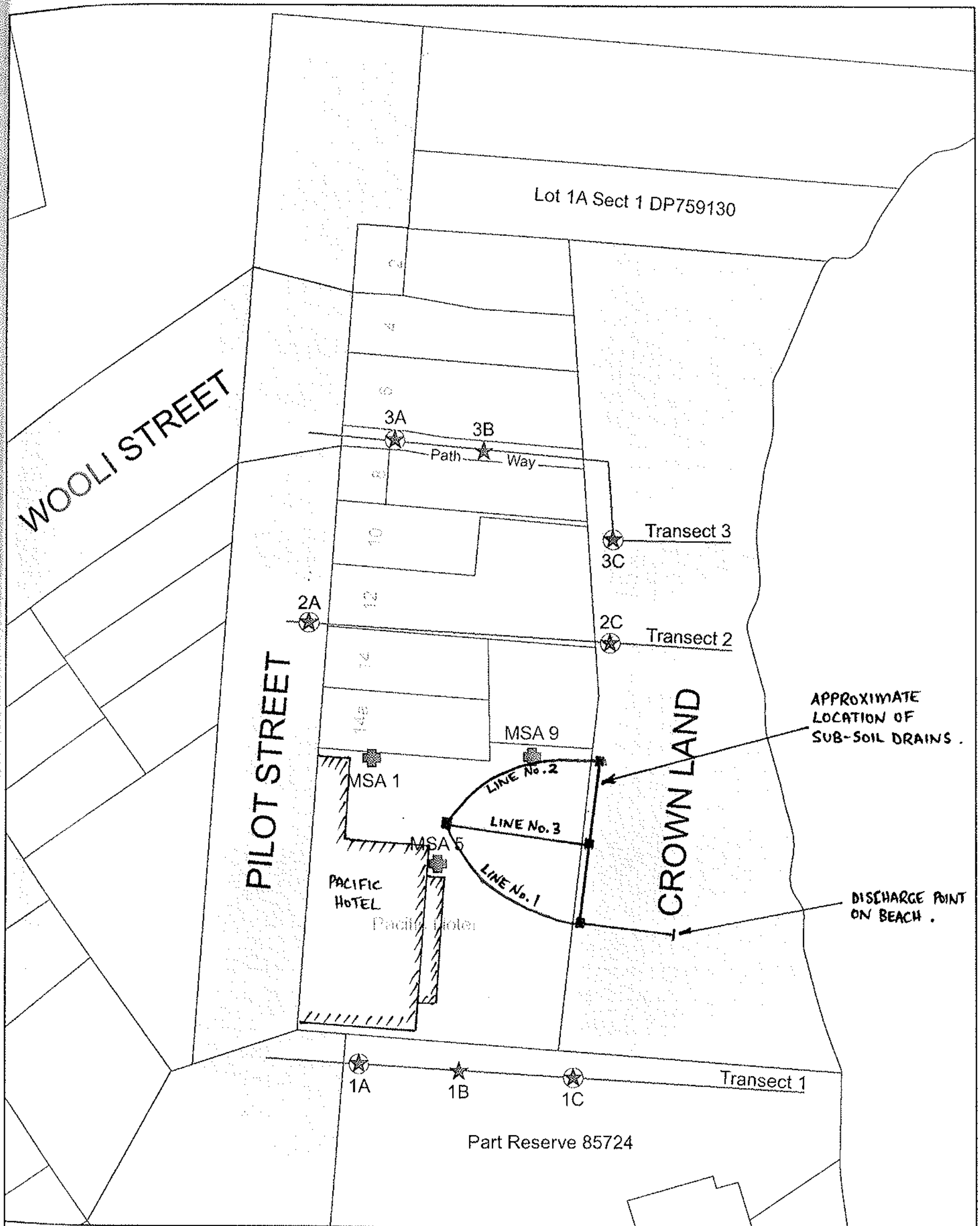
Report No. 19314WL2 Figure No. 2

WRL Report No. 2005/34	LOCATIONS OF PIEZOMETERS, INCLINOMETERS AND TRANSECTS 1 - 3	Figure 2



LEGEND:

- Inclinometer
- Piezometer
- Existing monitoring site

* LOCATION OF DRAINS HAS BEEN TAKEN FROM: DRAWING No. 99-629/09 REVISION B DATED 1/2/02 AS DRAWN BY MICHAEL SAMMS & ASSOC.

Map courtesy of Clarence Valley Council

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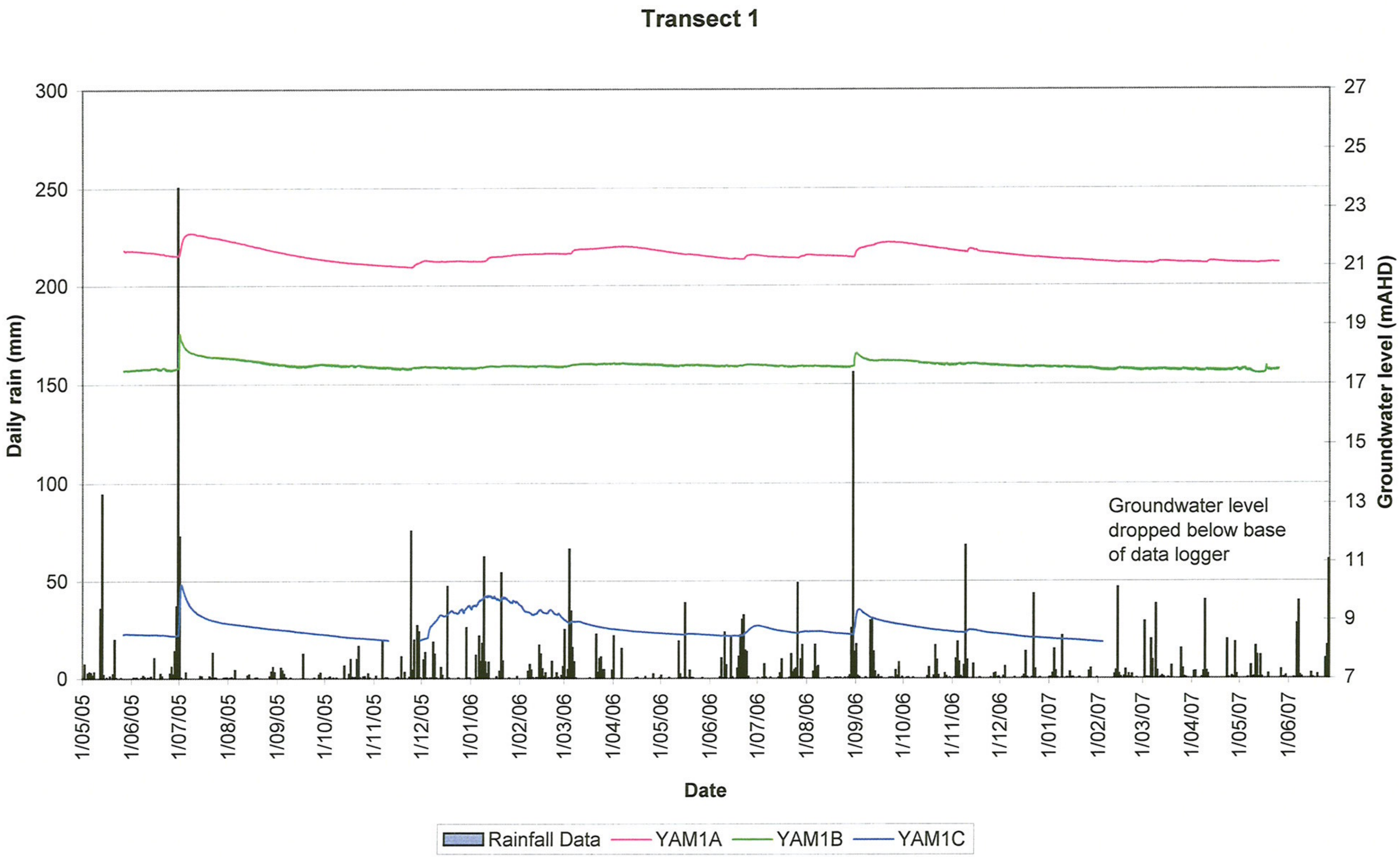
Report No. 19314WL2 Figure No. 3

WRL Report No. 2005/34	LOCATION OF SUB-SOIL DRAINS BELOW PACIFIC HOTEL	Figure 3



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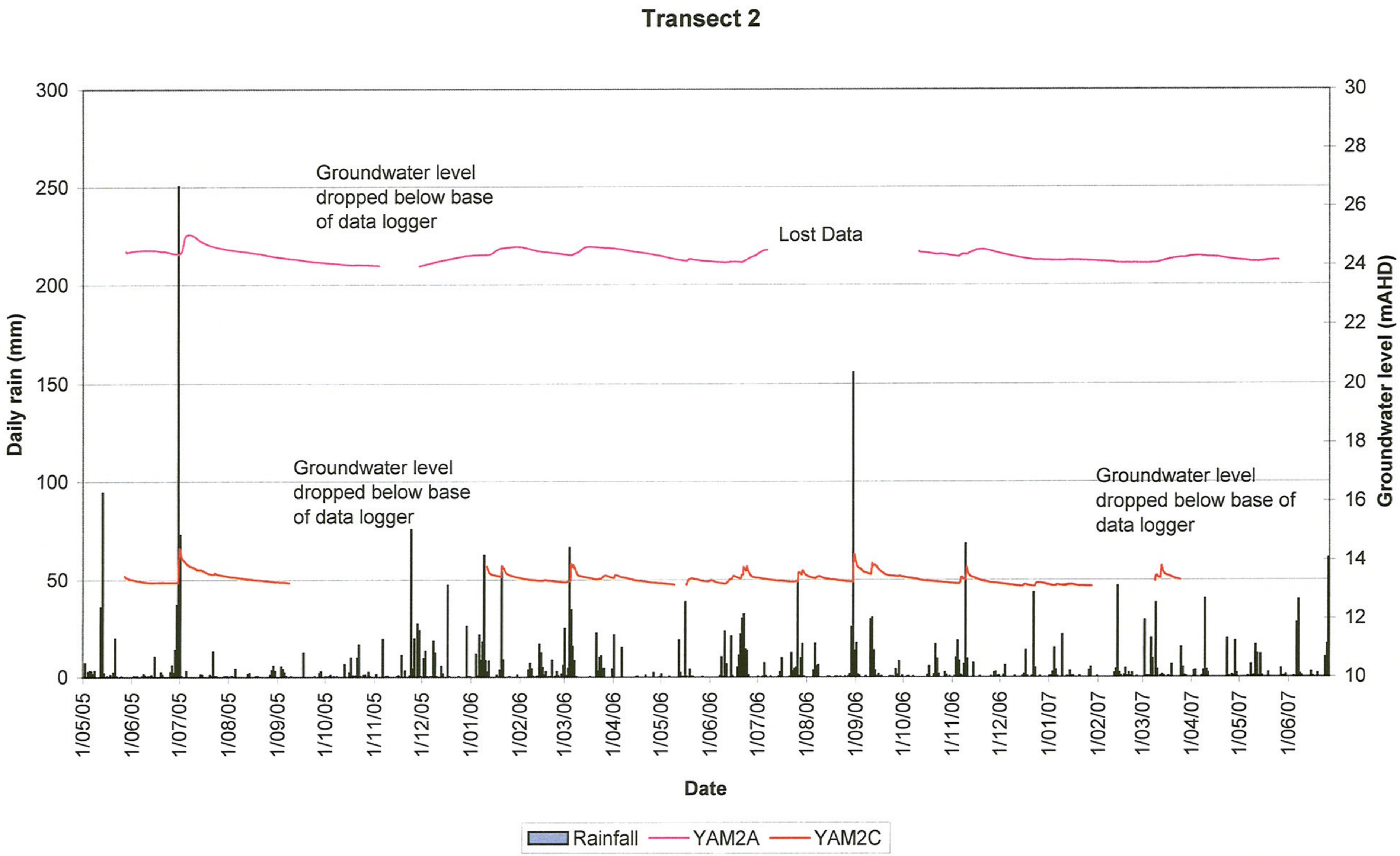
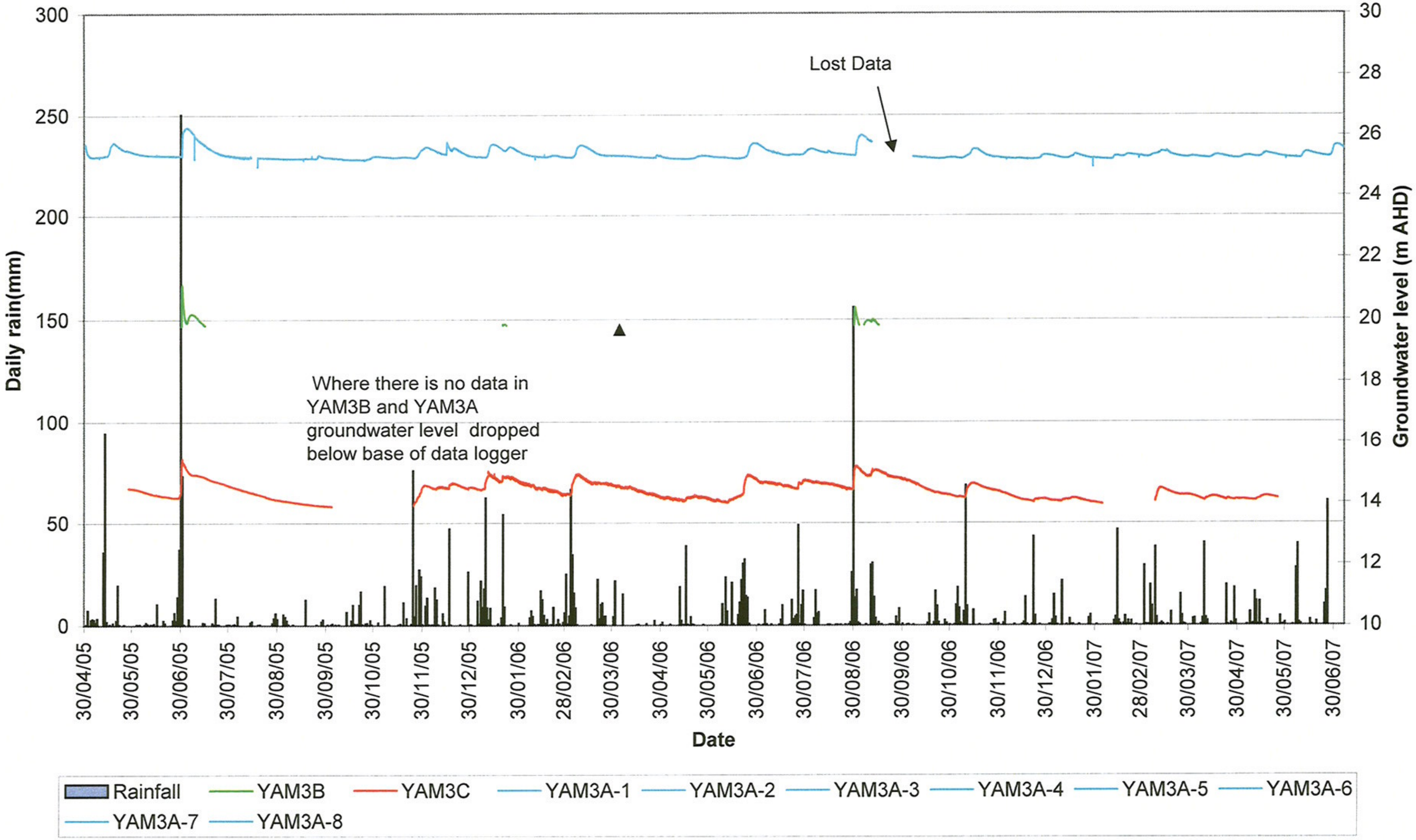
Report No. 19314WL2 Figure No. 4



Jeffery and Katauskas Pty Ltd	
Report No. 19314WL2	Figure No. 5

Transect 3



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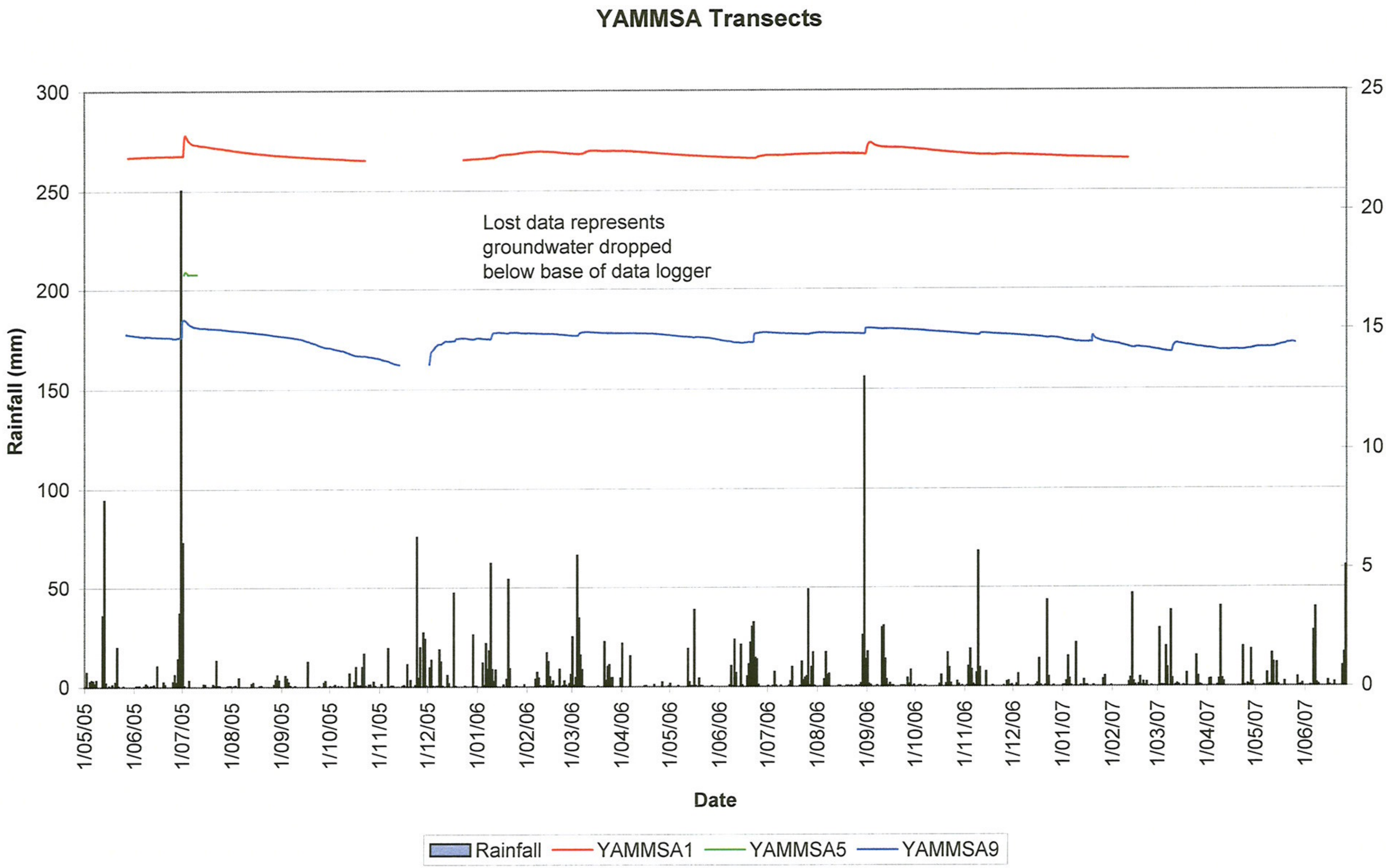
Report No. 19314WL2 Figure No. 6



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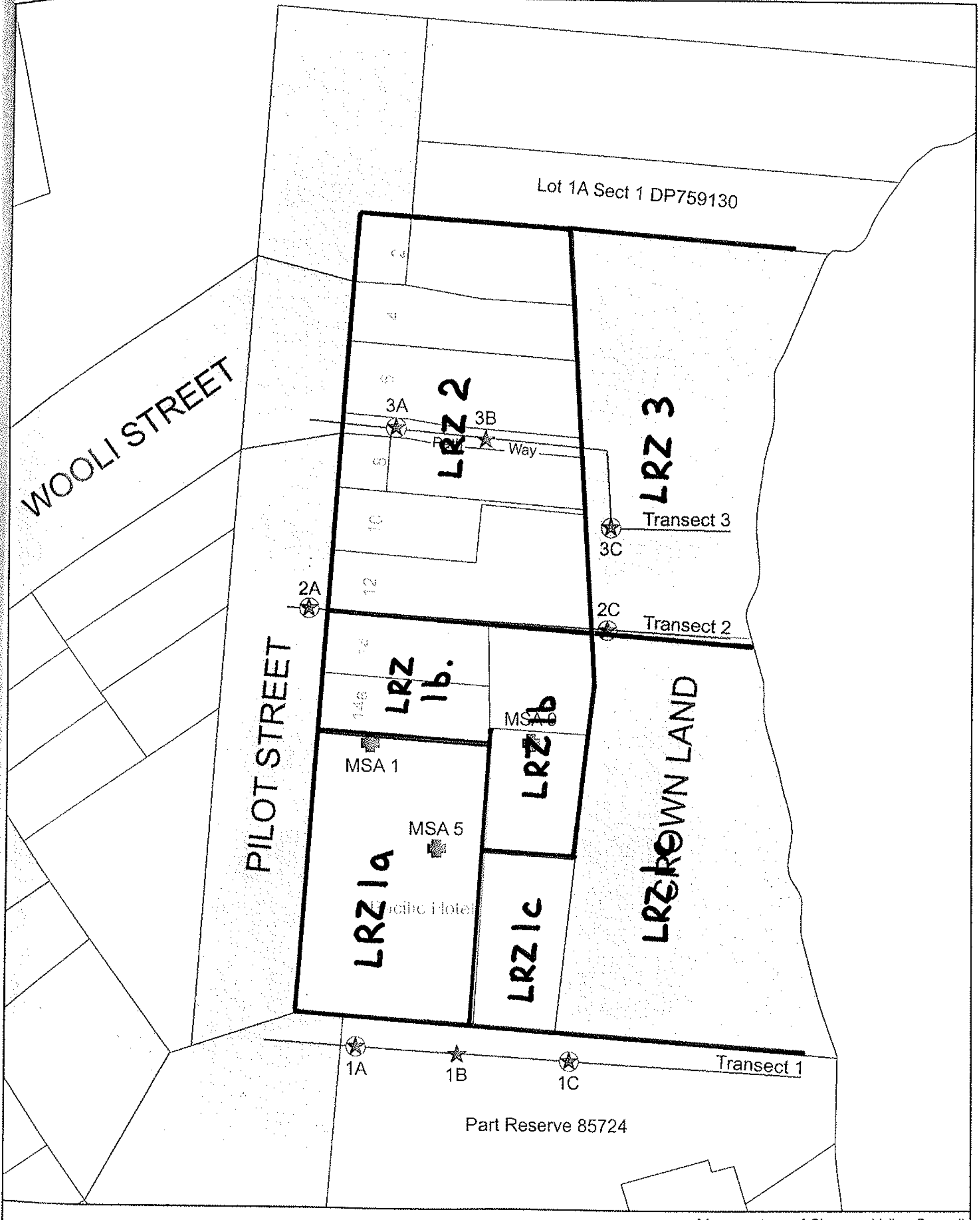
Report No. 19314WL2 Figure No. 7



Map courtesy of Clarence Valley Council

LEGEND:

- Inclinometer
- Piezometer
- Existing monitoring site

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Report No. 19314WL2 Figure No. 8

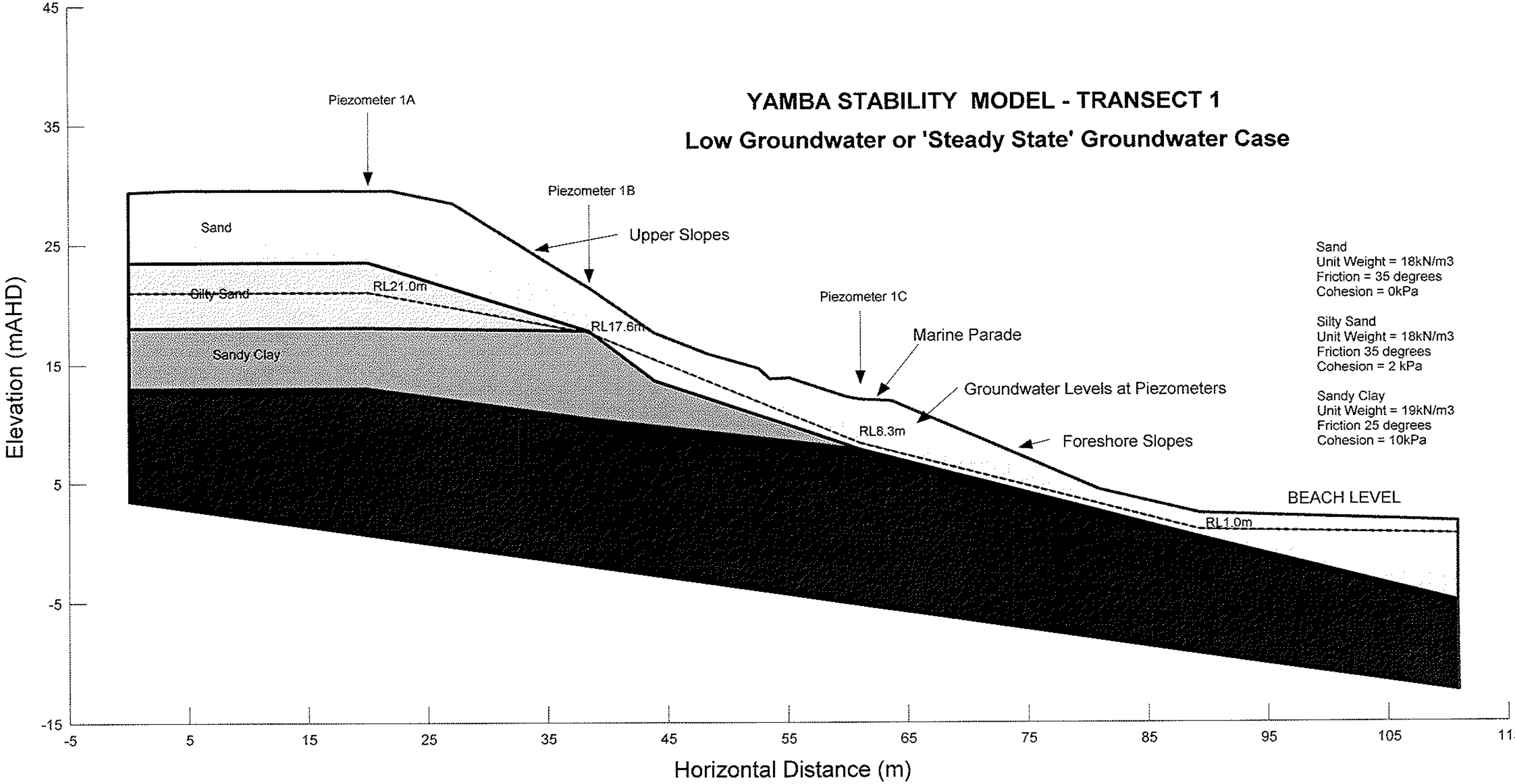
WRL Report No. 2005/34	LOCATION OF LANDSLIDE RISK ZONES	Figure 8

APPENDIX A

TABLE A1

SUMMARY TABLE OF SLIP CIRCLE RESULTS

Subsurface Model	Groundwater Level	Factor of Safety (Figure No.)	Failure Form
Transect 1	Low	1.1 (Figure A2) 1.2 (Figure A3) 1.8 (Figure A4)	Surficial Upper Slope Deep Seated Upper Slope Moderate Foreshore Slope
Transect 1	High	1.1 (Figure A6) 1.4 (Figure A7)	Deep Seated Upper Slope Deep Seated Foreshore Slope
Transect 1 With Upper Bound Soil Parameters	High	1.2 (Figure A9) 1.7 (Figure A10)	Deep Seated Upper Slope Deep Seated Foreshore Slope
Transect 1 With Additional 0.5m Groundwater Rise	High	1.0 (Figure A12)	Deep Seated Upper Slope
Transect 1 With YAMMSA1, YAMMSA5 and YAMMSA9	Low	1.1 (Figure A14) 1.4 (Figure A15) 1.8 (Figure A16)	Surficial Upper Slope Deep Seated Upper Slope Moderate Foreshore Slope
Transect 1 With YAMMSA1, YAMMSA5 and YAMMSA9	High	1.1 (Figure A18) 1.3 (Figure A19) 1.8 (Figure A20)	Surficial Upper Slope Deep Seated Upper Slope Moderate Foreshore Slope
Transect 2	Low	1.2 (Figure A22) 1.5 (Figure A23) 1.4 (Figure A24)	Surficial Upper Slope Moderate Upper Slope Deep Seated Foreshore Slope
Transect 2	High	1.2 (Figure A26) 1.2 (Figure A27) 1.5 (Figure A28)	Surficial Upper Slope Deep Seated Foreshore Slope Moderate Upper Slope
Transect 3	Low	1.2 (Figure A30) 2.1 (Figure A31)	Deep Seated Foreshore Slope Deep Seated Upper Slope
Transect 3	High	0.8 (Figure A33) 1.6 (Figure A34) 2.0 (Figure A35)	Deep Seated Foreshore Slope Moderate Toe Upper Slope Deep Seated Upper Slope



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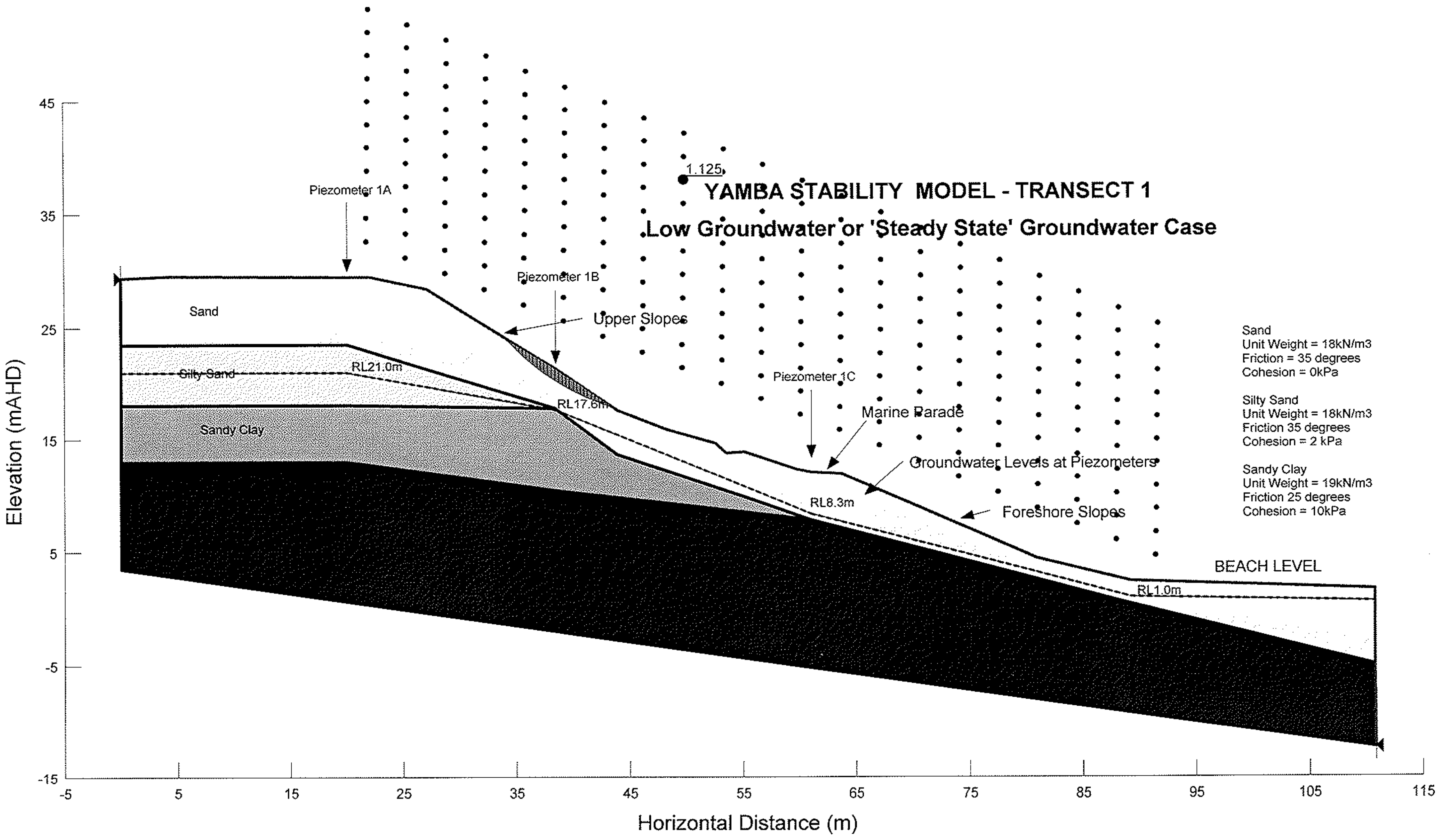
Report No. 19314WL2 Figure No. A1



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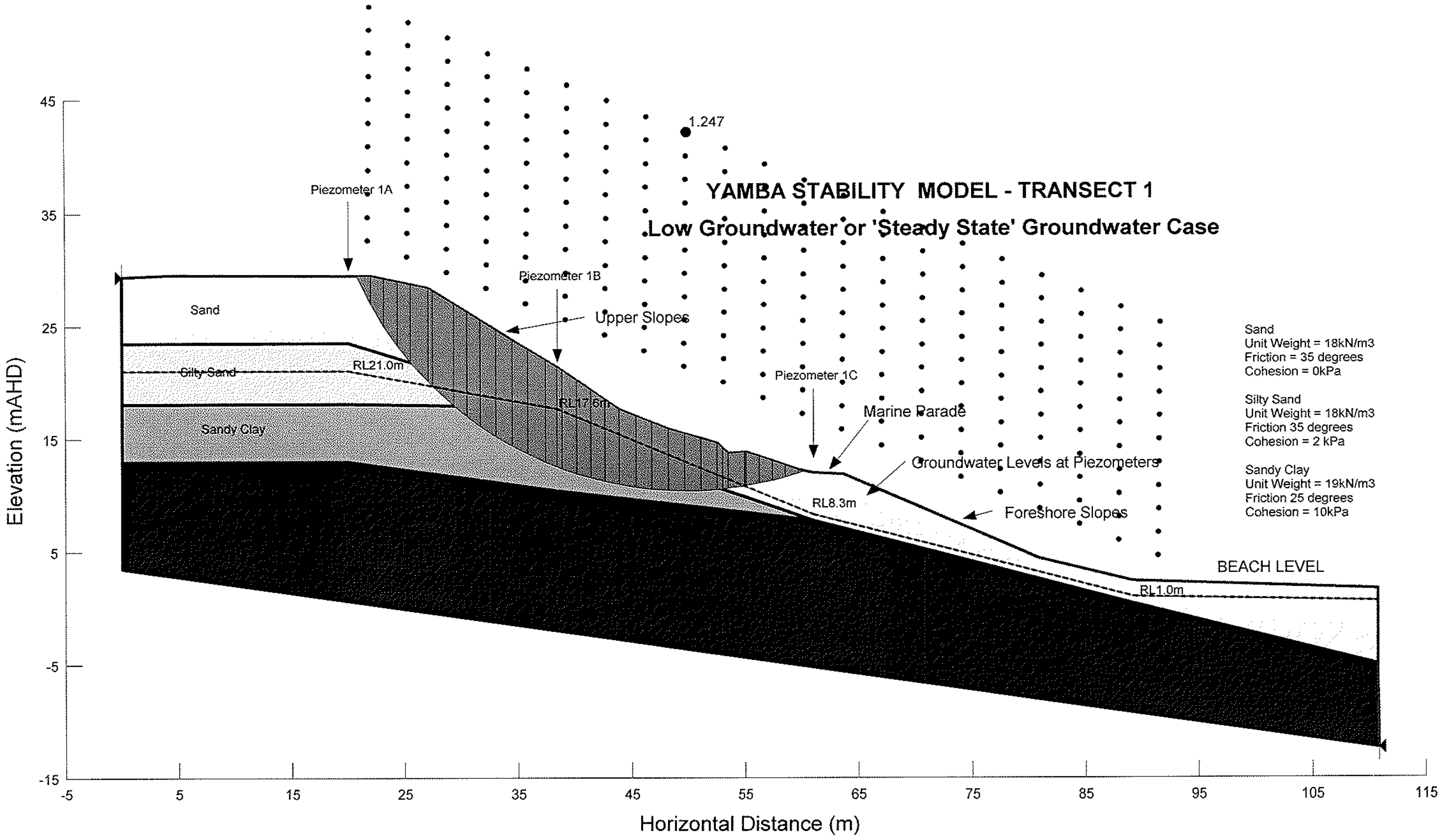
Report No. 19314WL2 Figure No. A2



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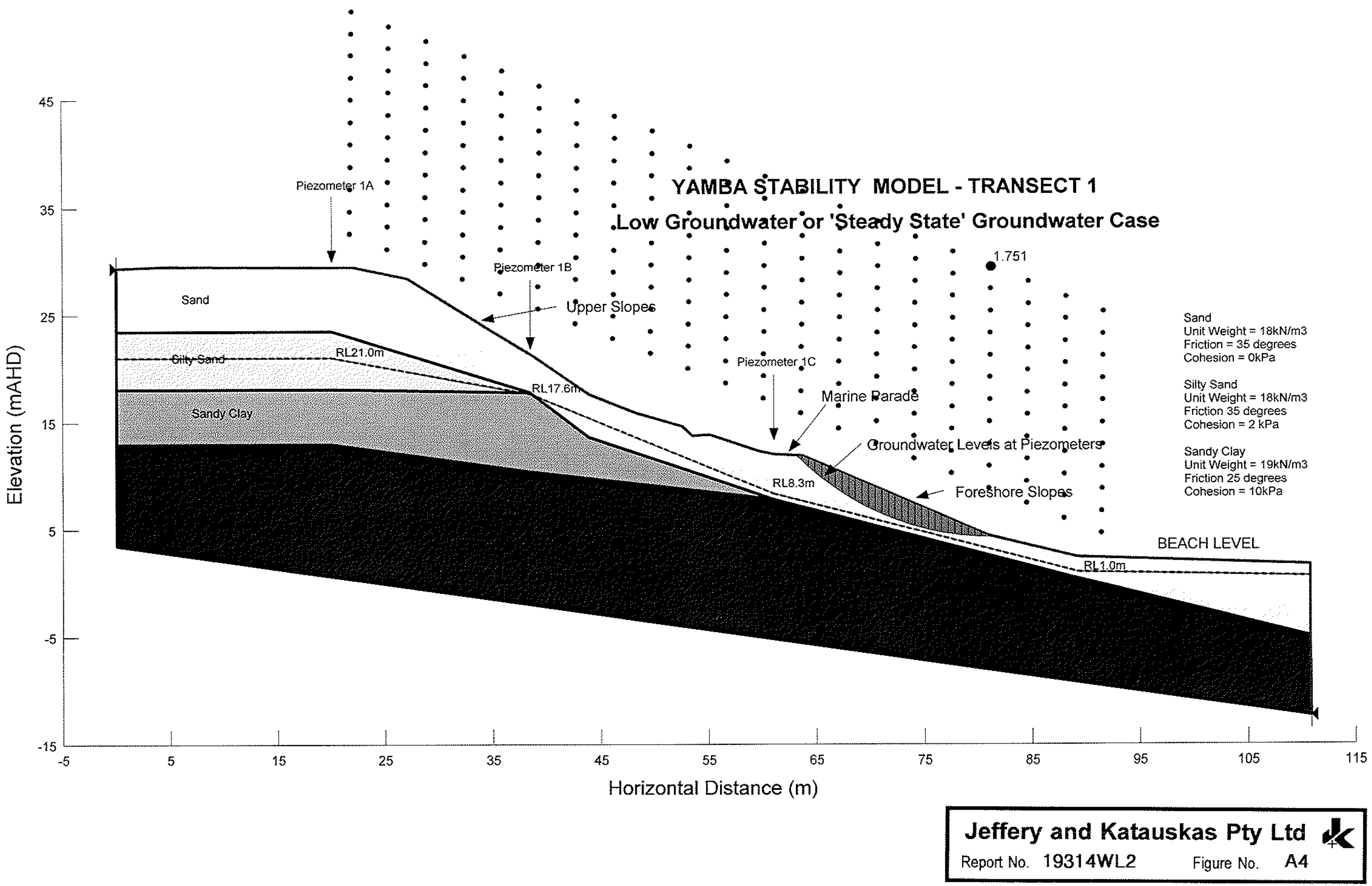
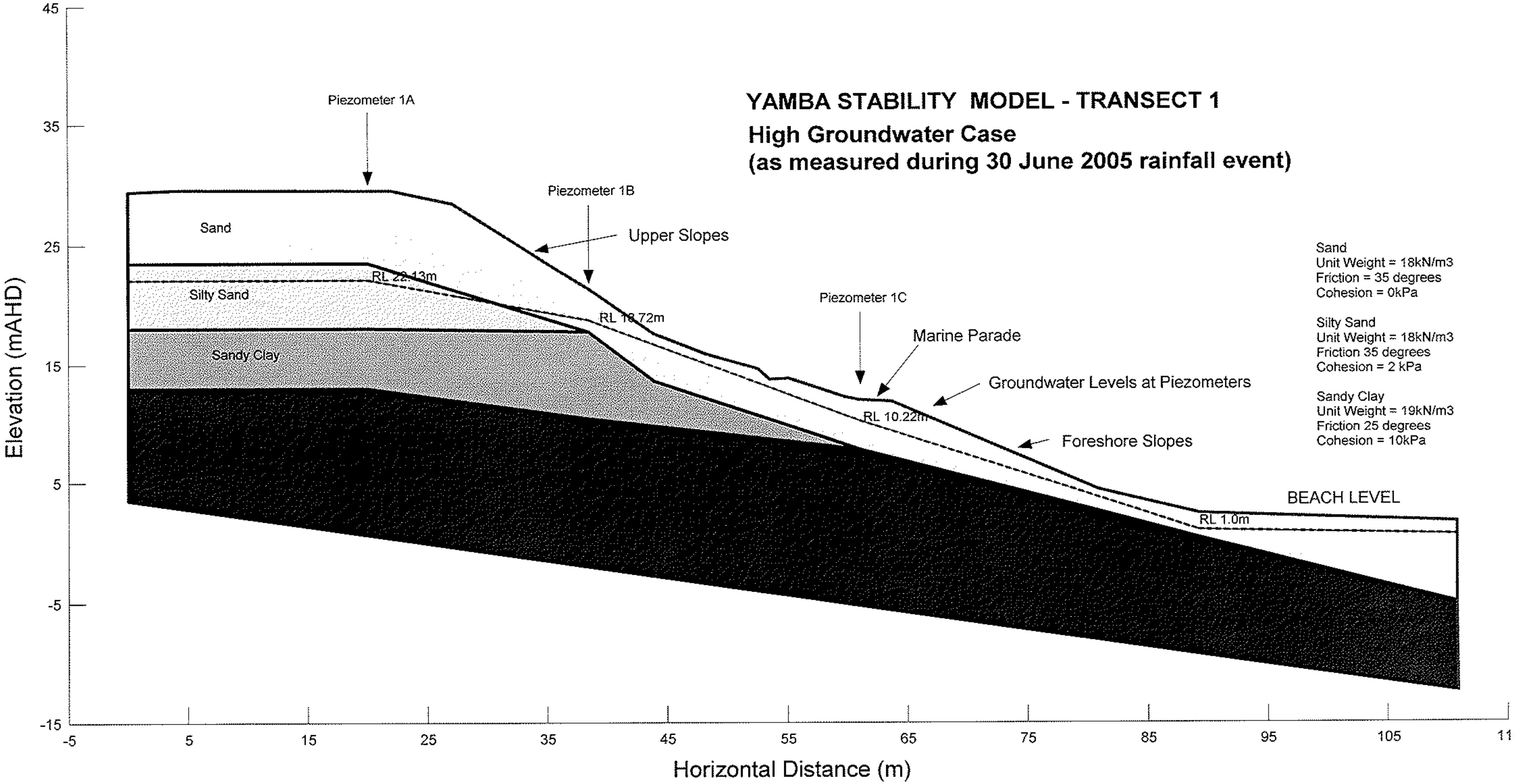
Report No. 19314WL2 Figure No. A3


YAMBA STABILITY MODEL - TRANSECT 1
Low Groundwater or 'Steady State' Groundwater Case
Piezometer 1A
Piezometer 1B
Piezometer 1C
Upper Slopes
Marine Parade
Groundwater Levels at Piezometers
Foreshore Slopes
BEACH LEVEL
1.751
Sand
Silty Sand
Sandy Clay
RL21.0m
RL17.6m
RL8.3m
RL1.0m
Sand
Unit Weight = 18kN/m3
Friction = 35 degrees
Cohesion = 0kPa
Silty Sand
Unit Weight = 18kN/m3
Friction 35 degrees
Cohesion = 2 kPa
Sandy Clay
Unit Weight = 19kN/m3
Friction 25 degrees
Cohesion = 10kPa
Elevation (mAHD)
Horizontal Distance (m)
45
35
25
15
5
-5
-15
-5
5
15
25
35
45
55
65
75
85
95
105
115
Jeffery and Katauskas Pty Ltd
Report No. 19314WL2
Figure No. A4



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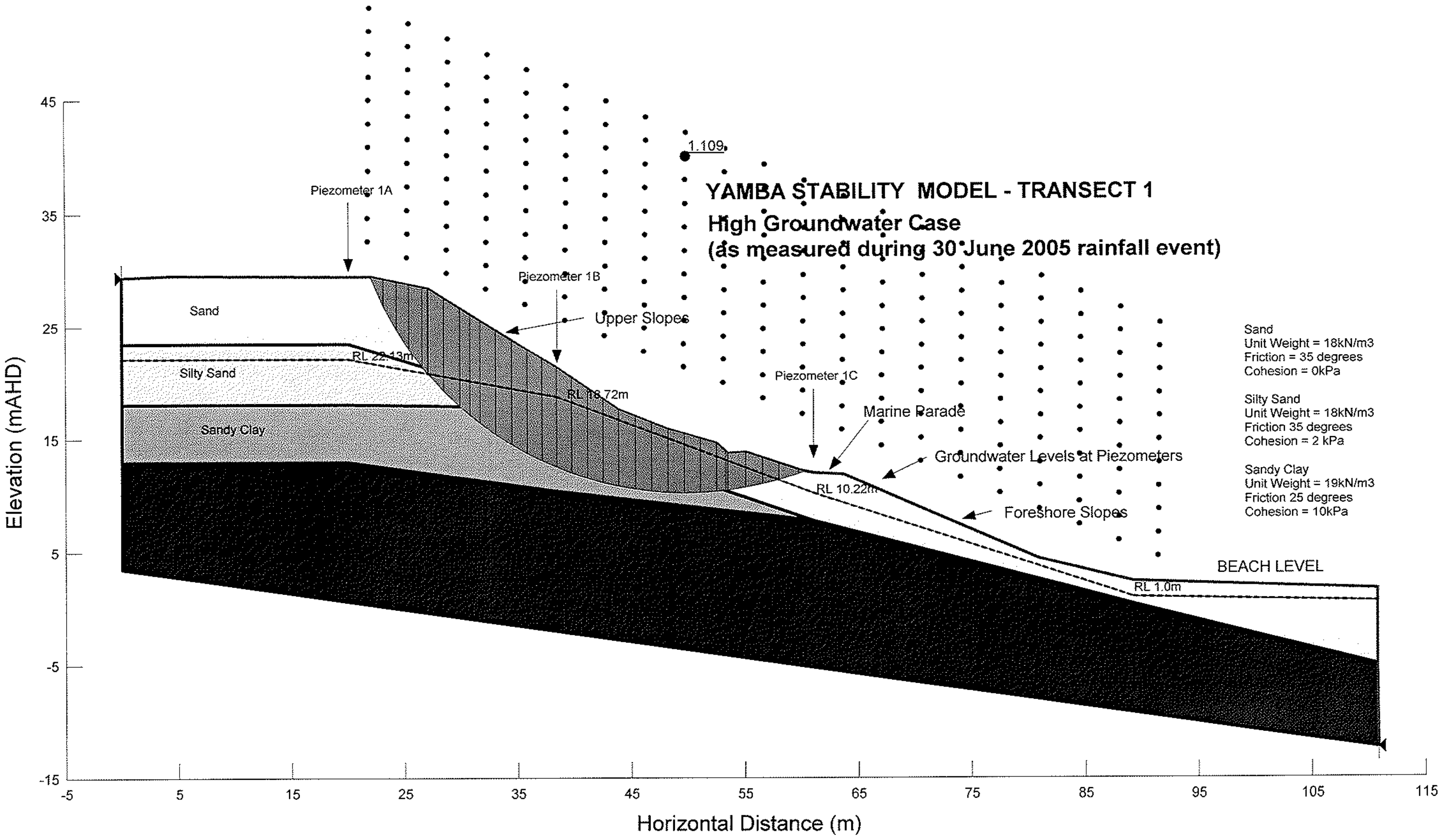
Report No. 19314WL2 Figure No. A5



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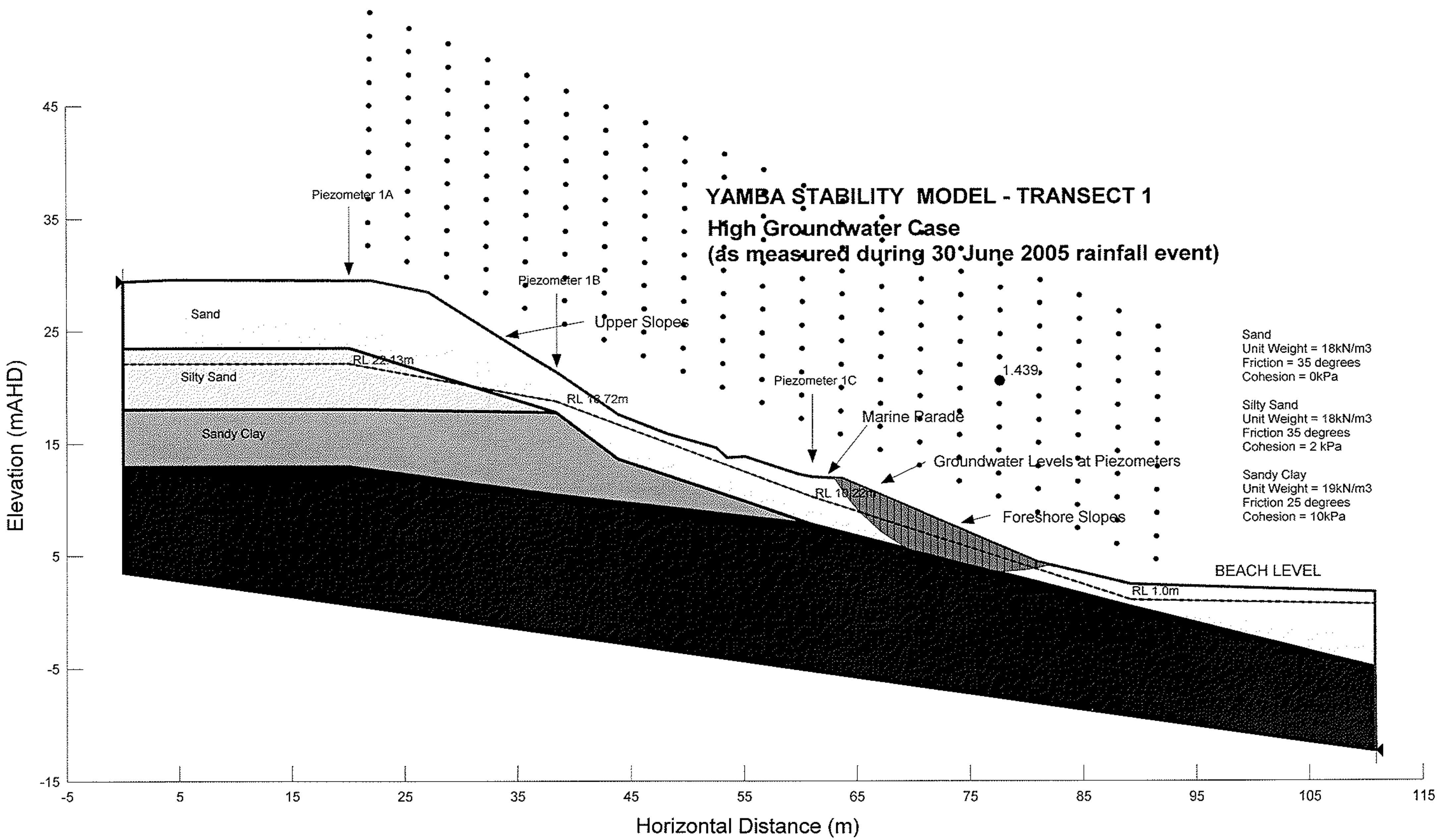
Report No. 19314WL2 Figure No. A6



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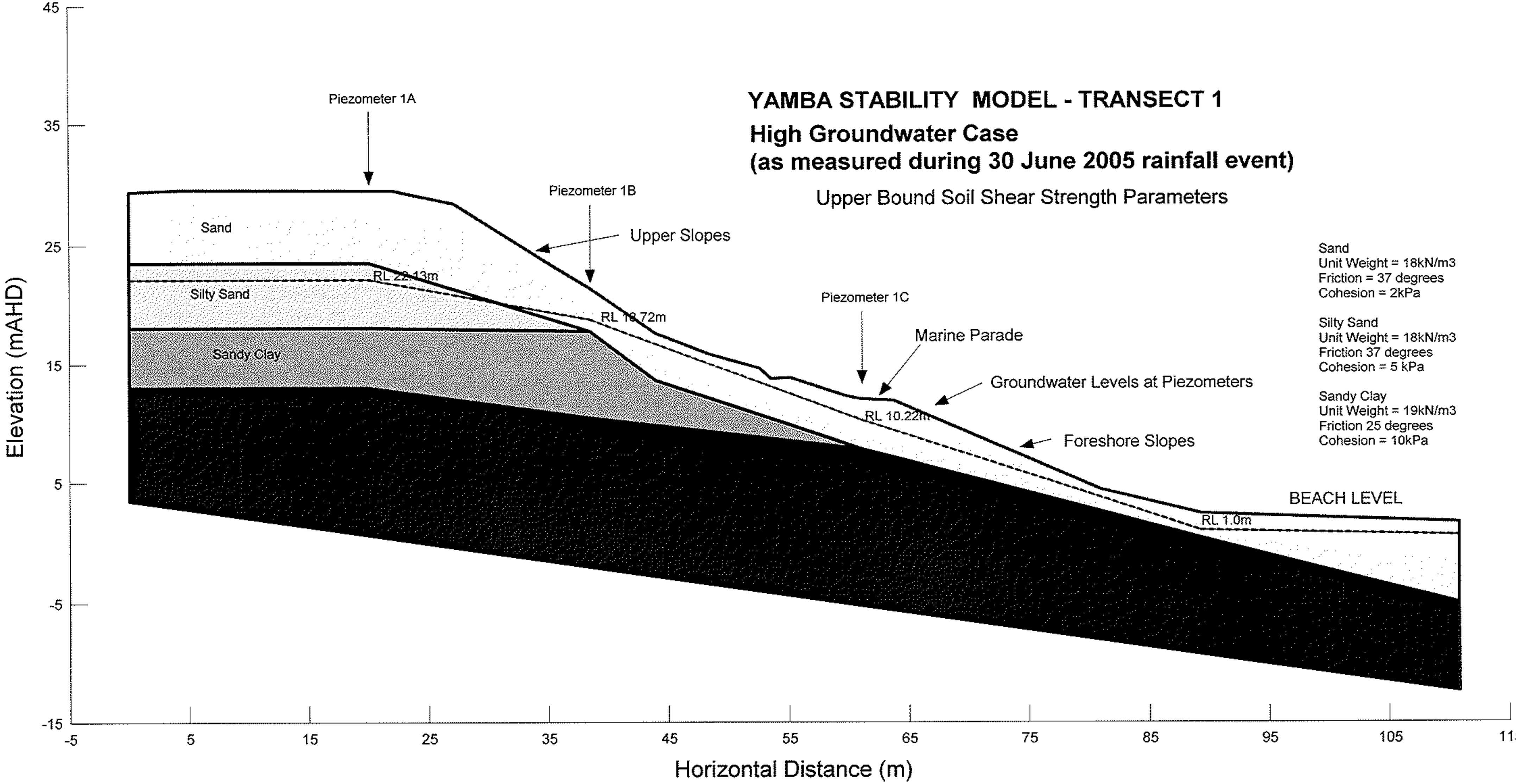
Report No. 19314WL2 Figure No. A7



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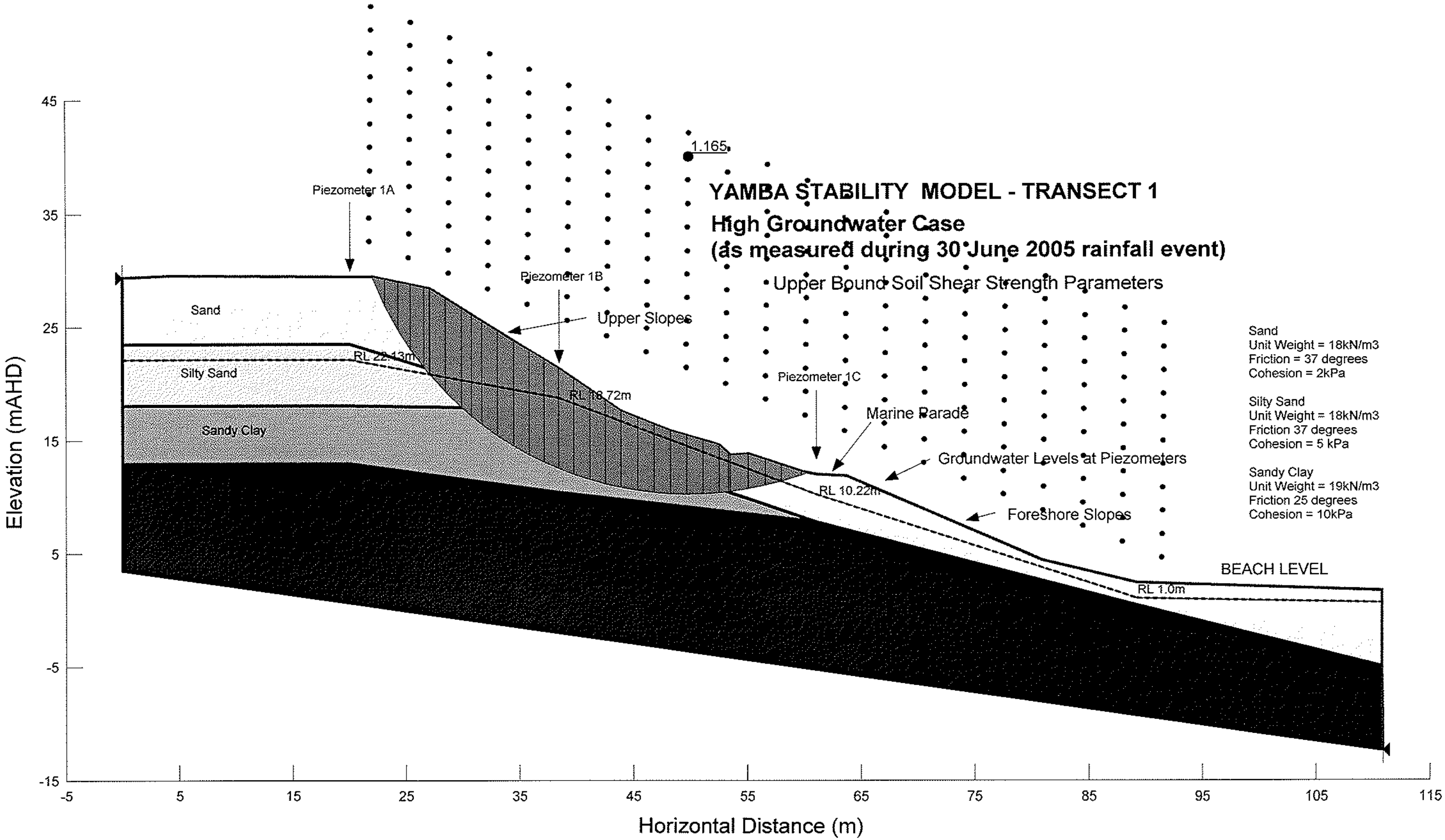
Report No. 19314WL2 Figure No. A8



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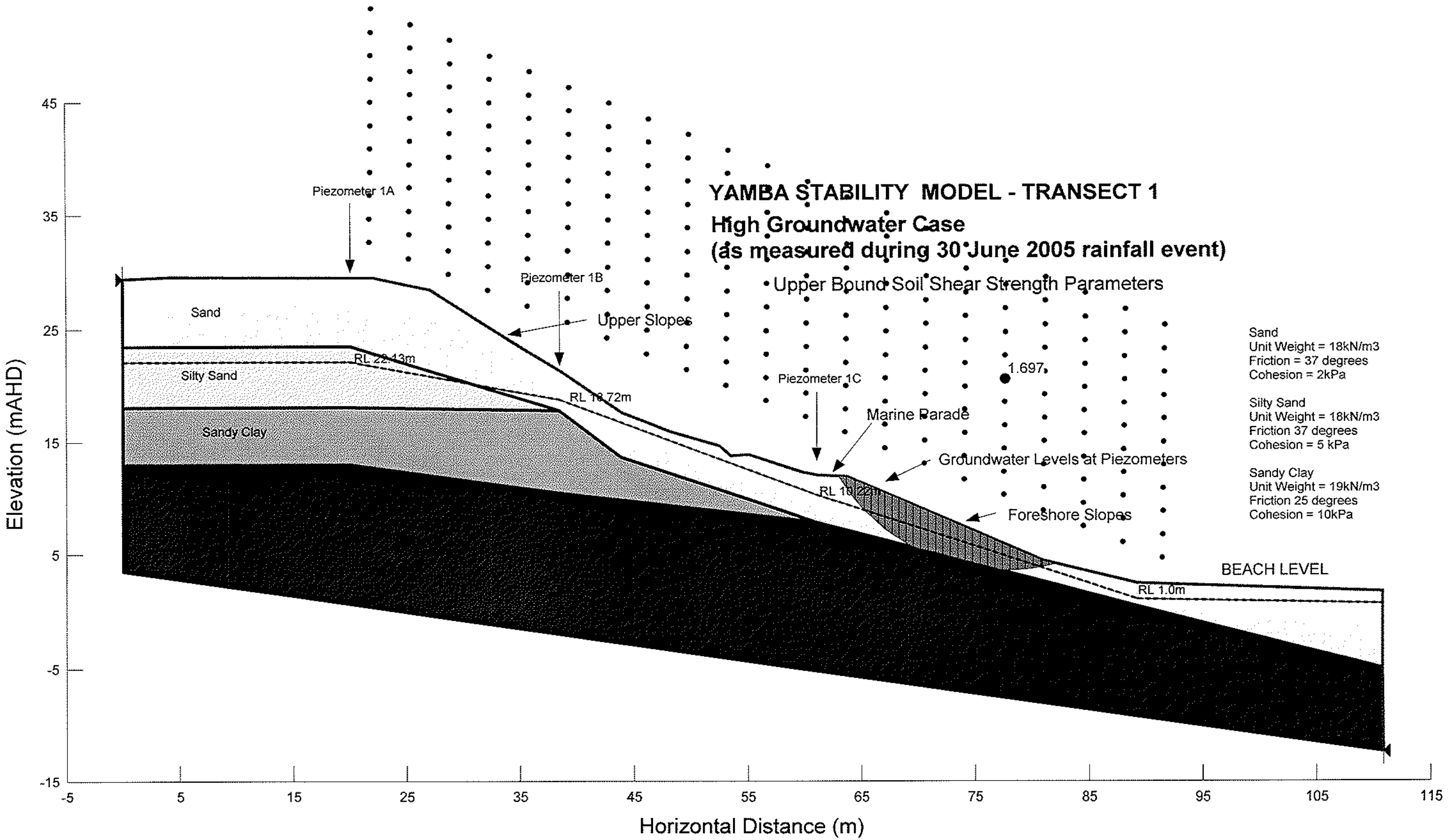
Report No. 19314WL2 Figure No. A9



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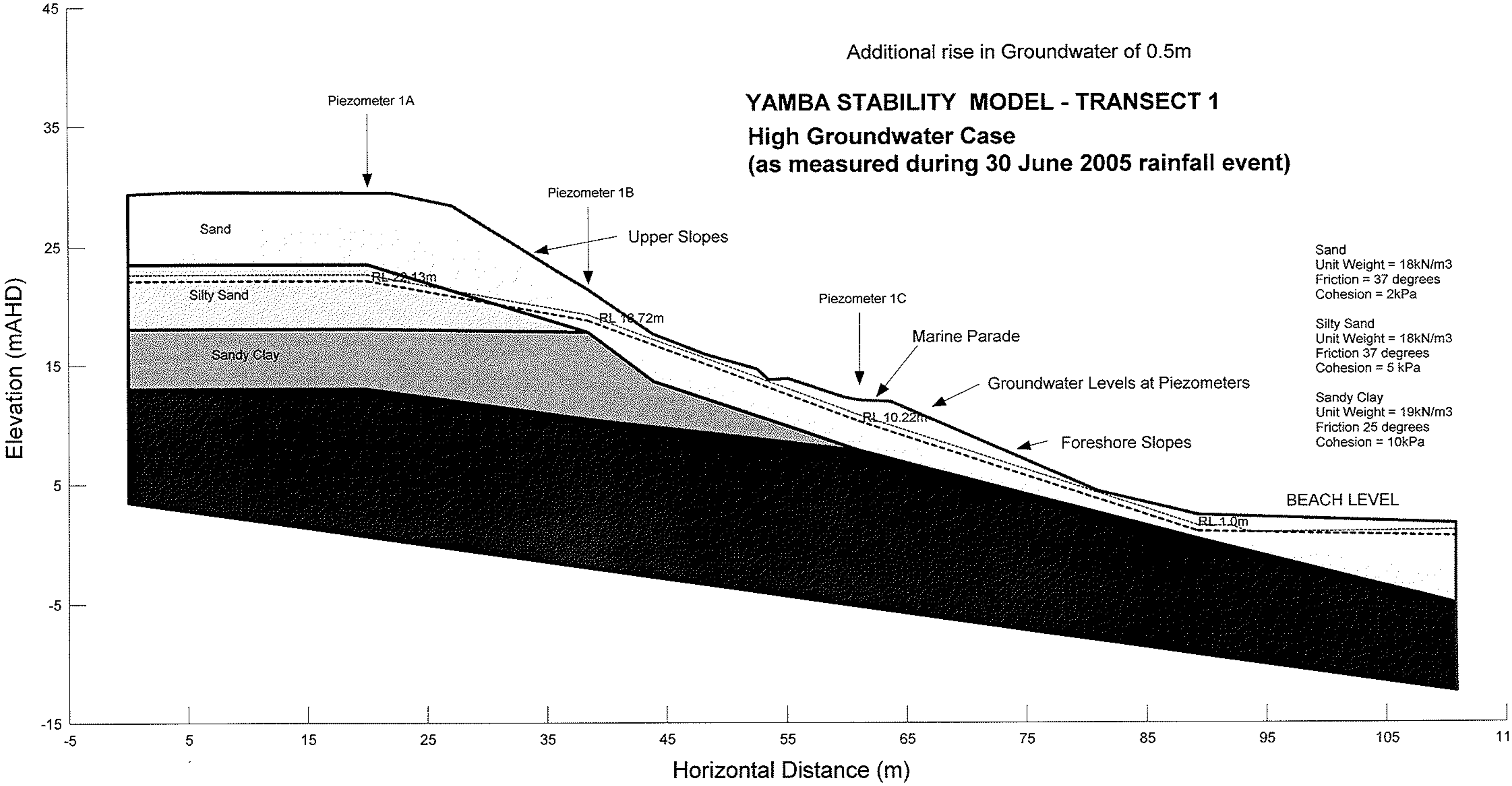
Report No. 19314WL2 Figure No. A10



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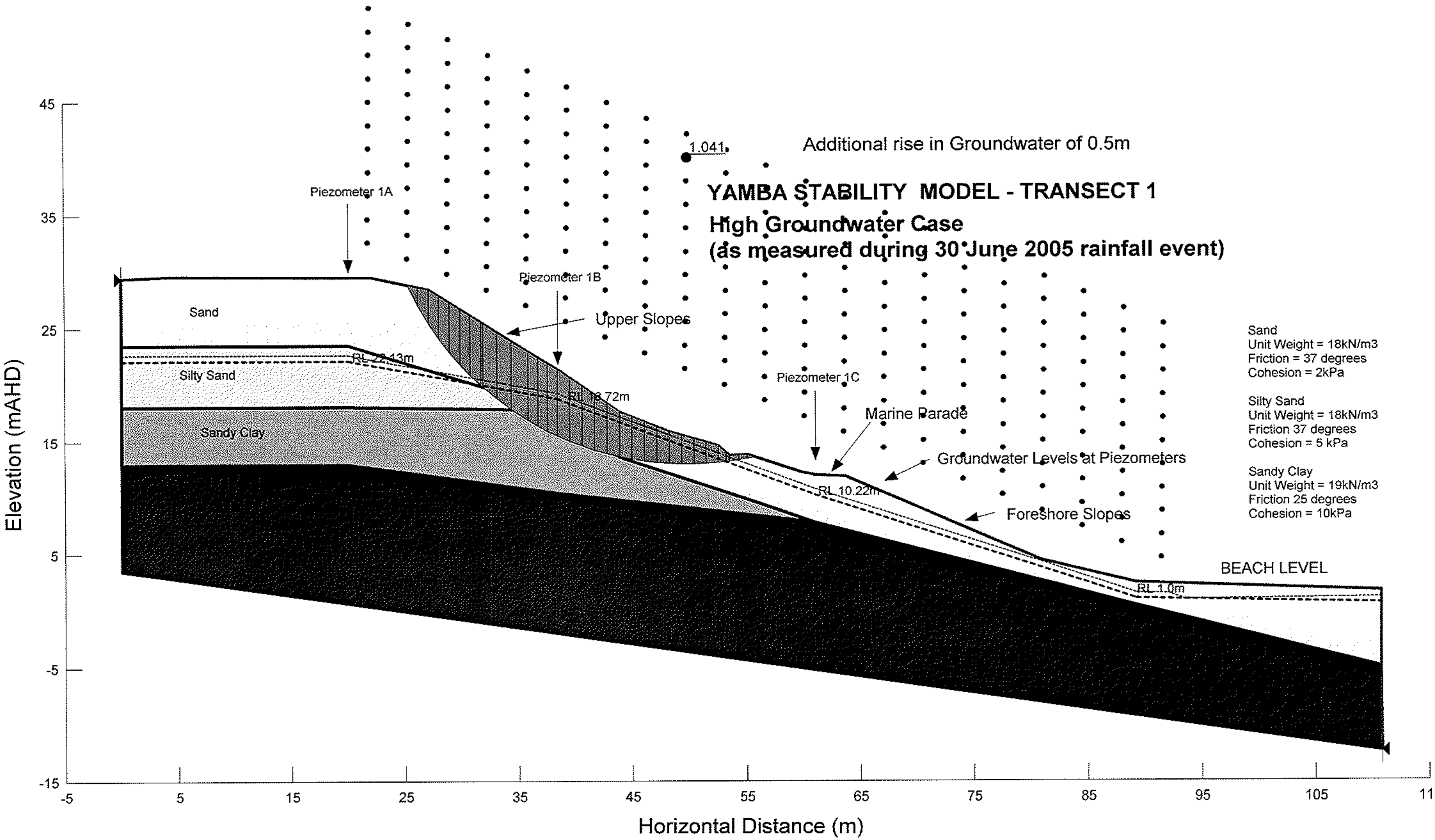
Report No. 19314WL2 Figure No. A11



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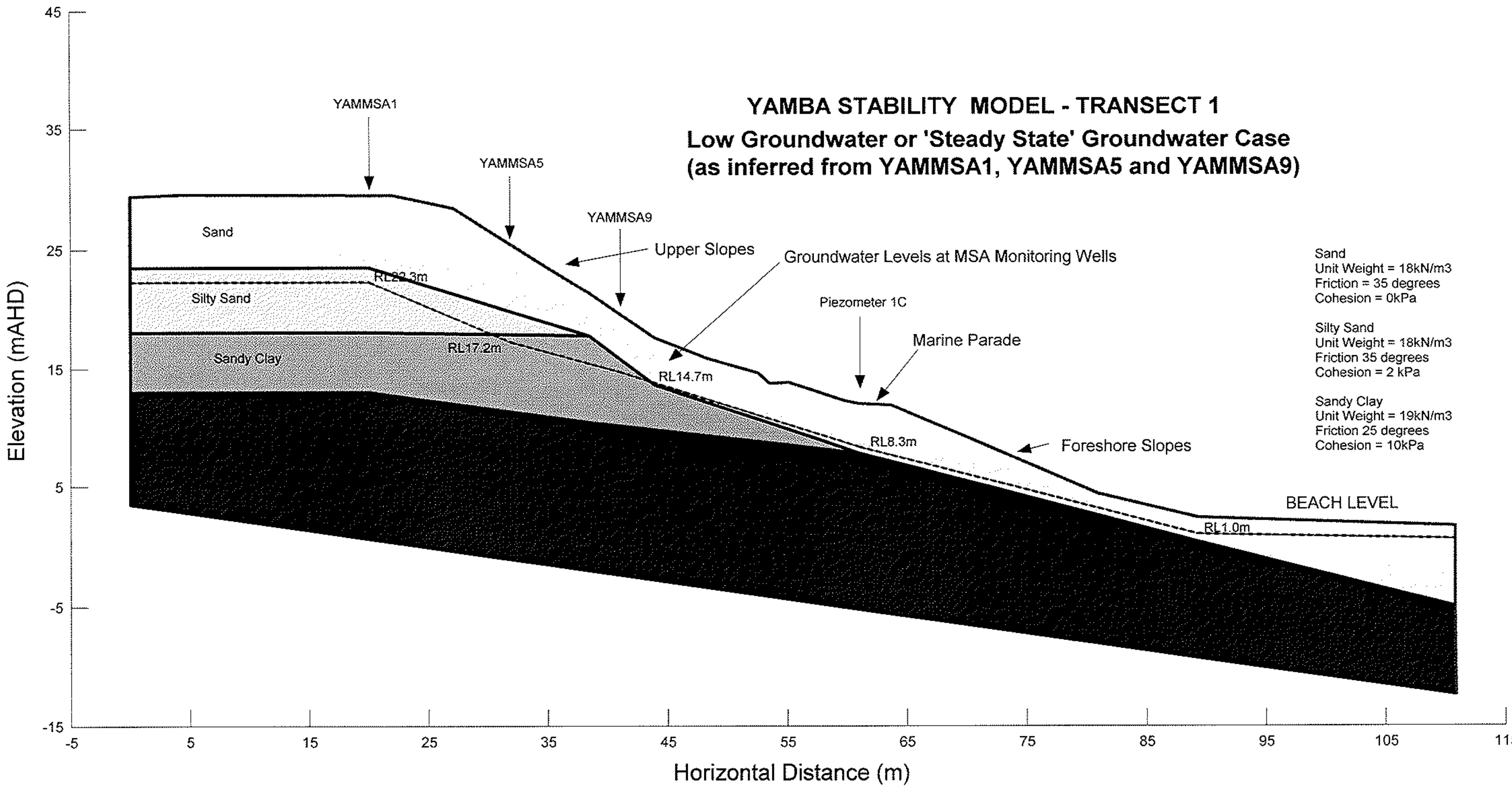
Report No. 19314WL2 Figure No. A12



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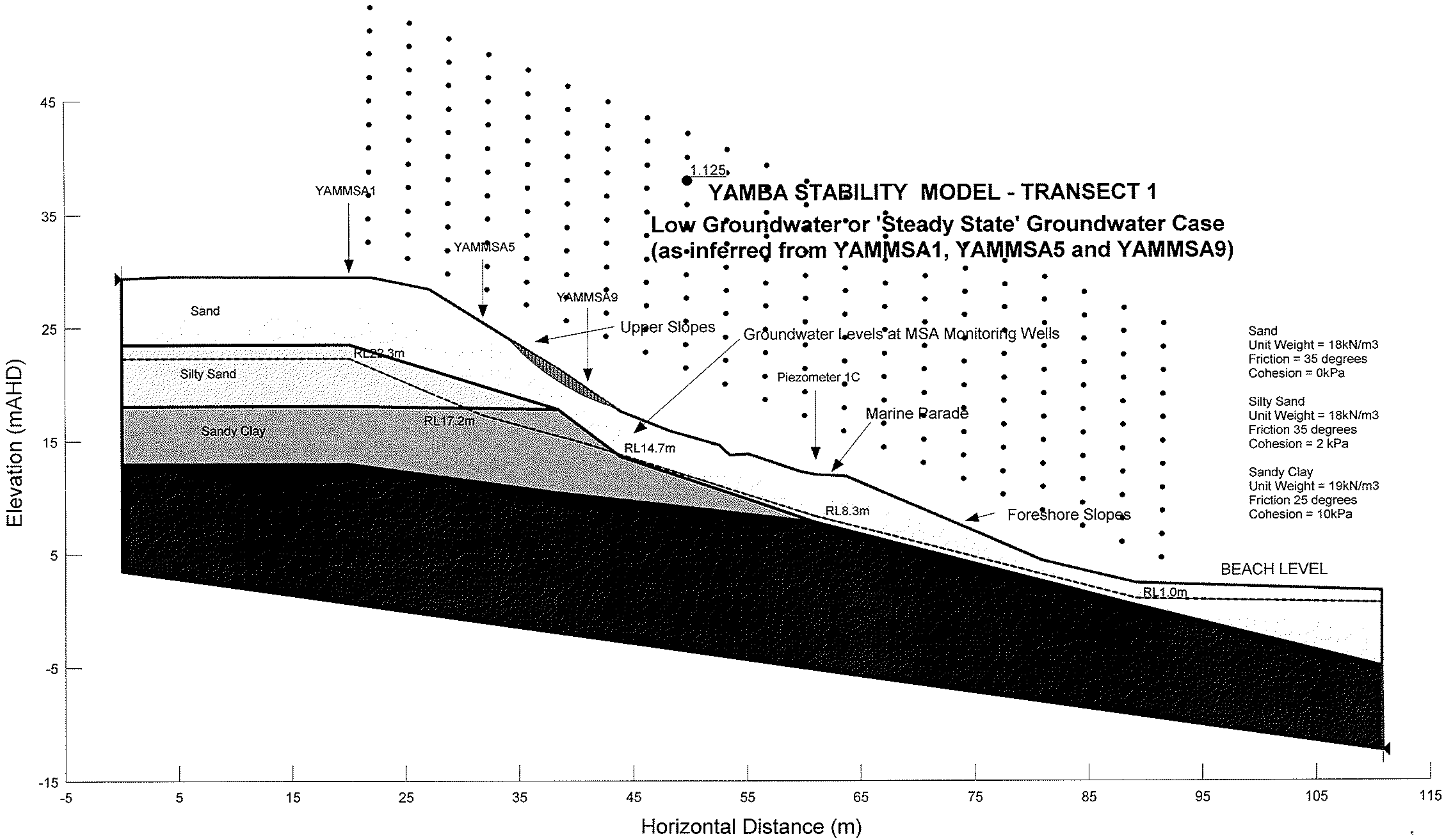
Report No. 19314WL2 Figure No. A13



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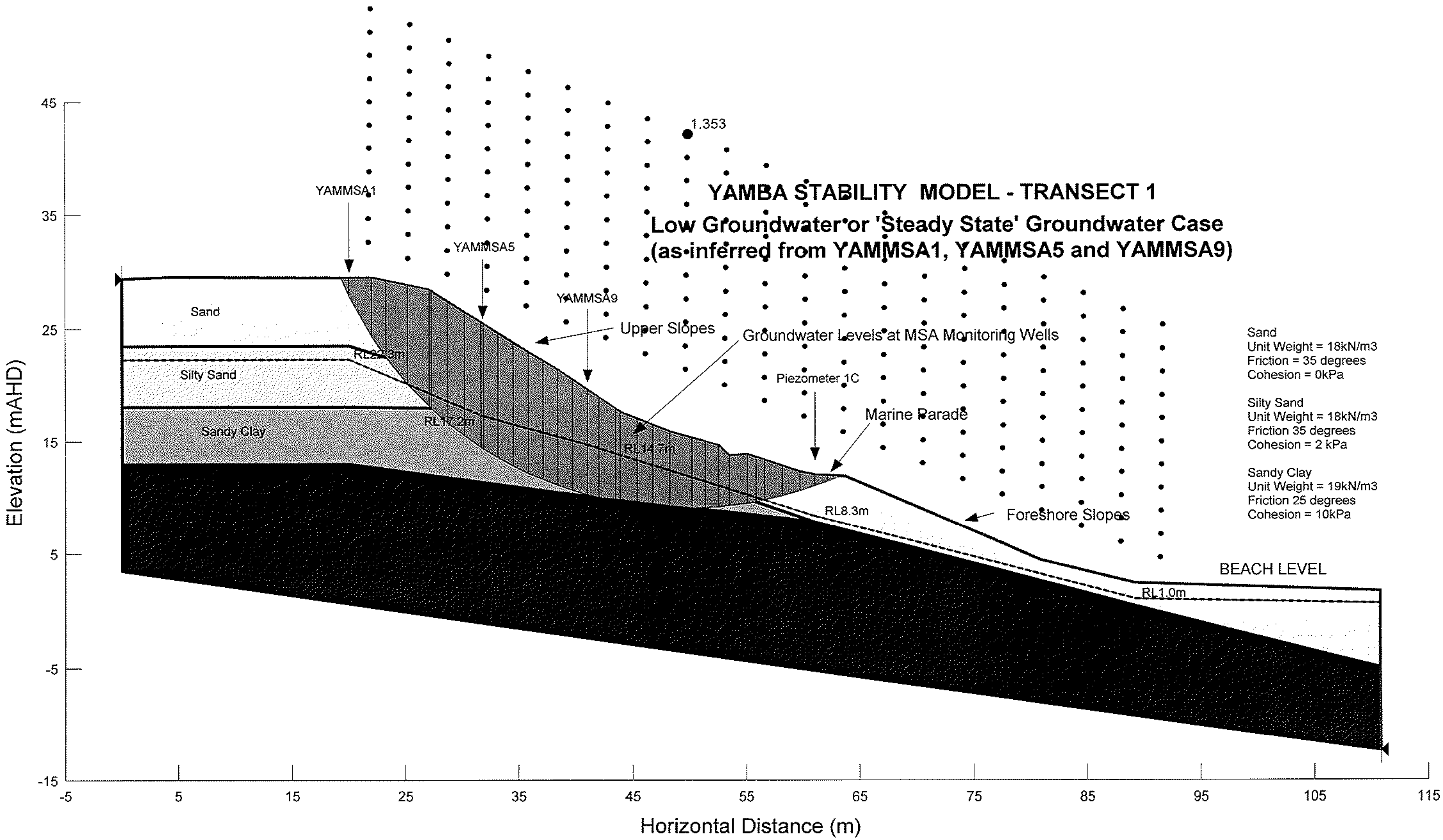

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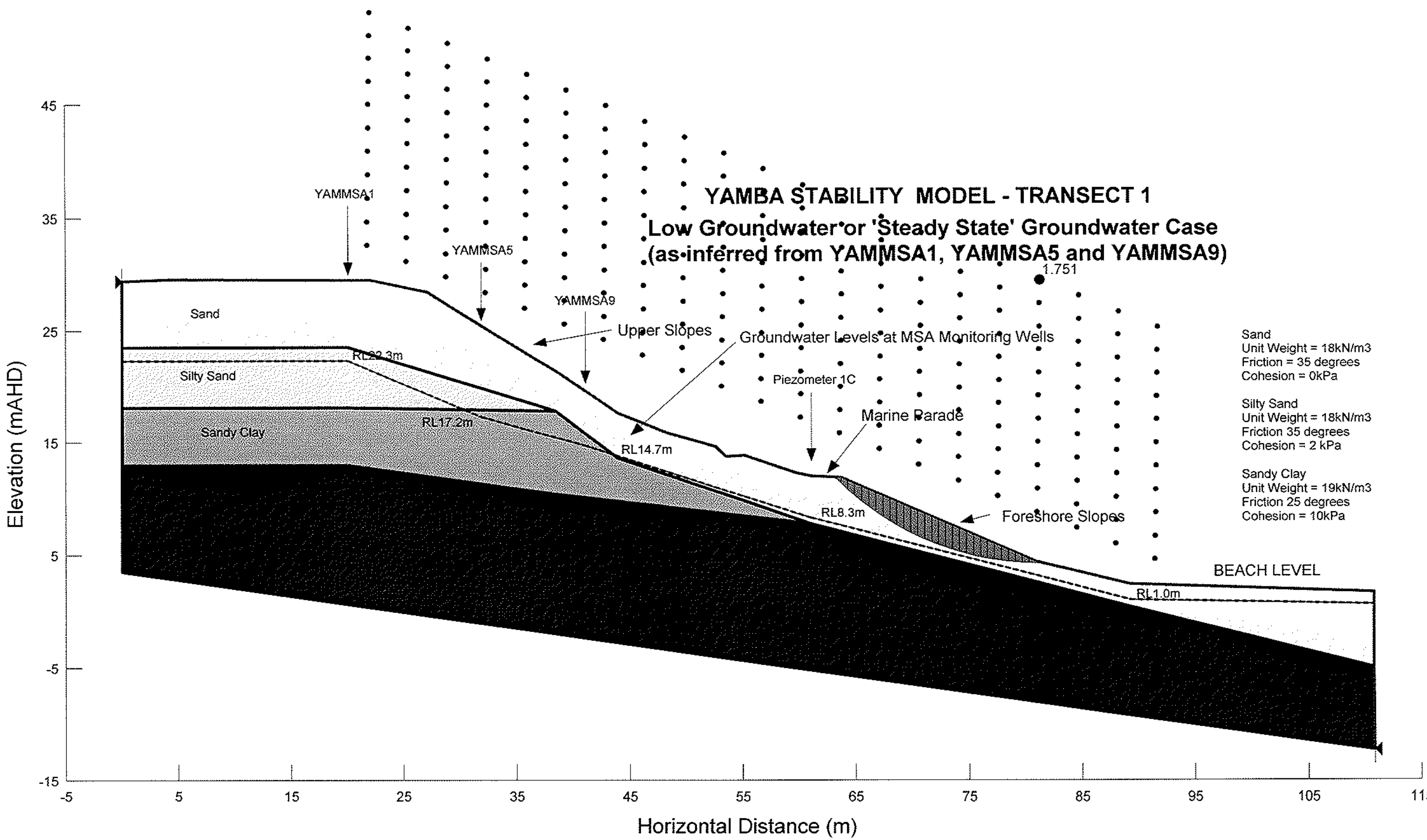
Report No. 19314WL2 Figure No. A15



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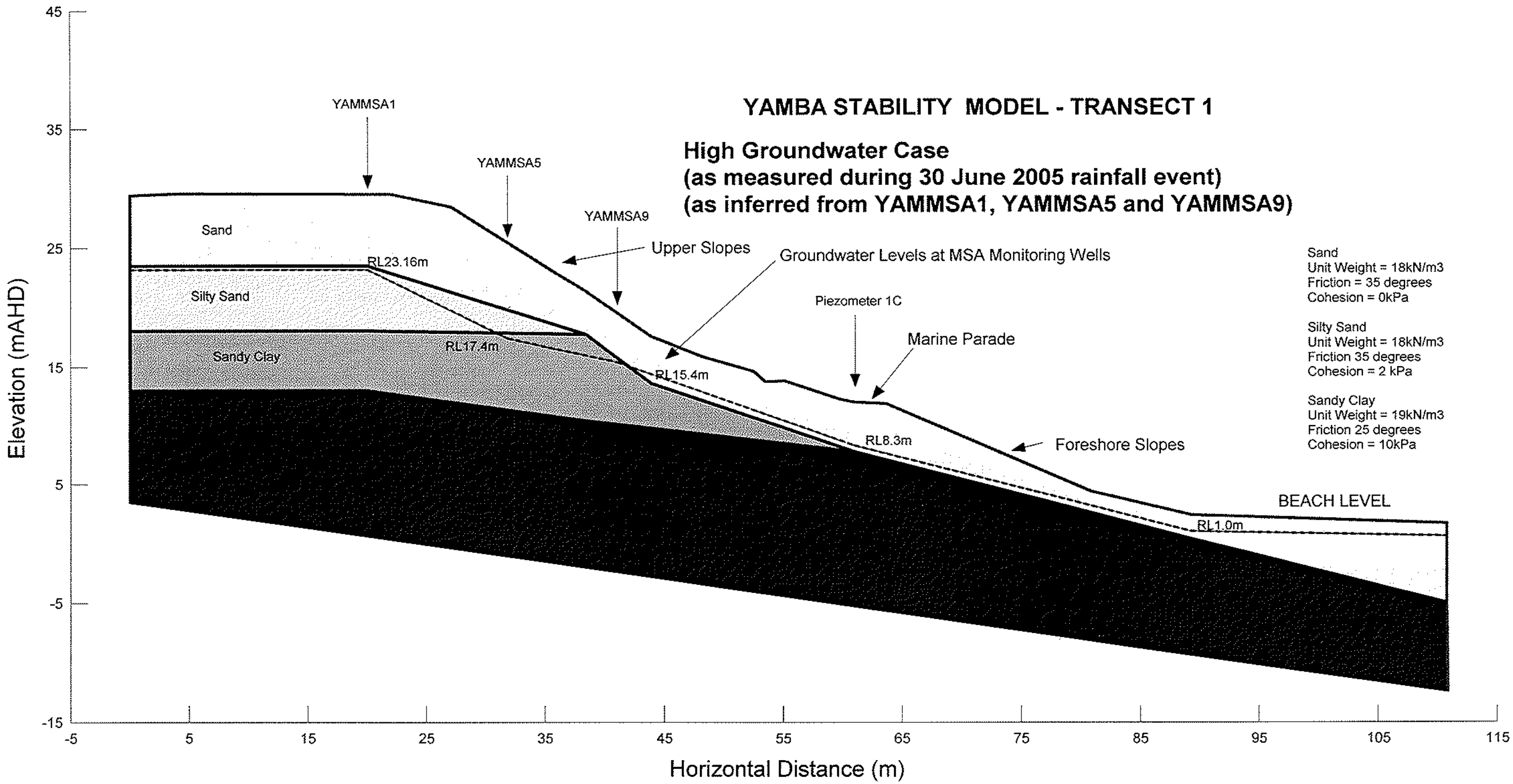
Report No. 19314WL2 Figure No. A16



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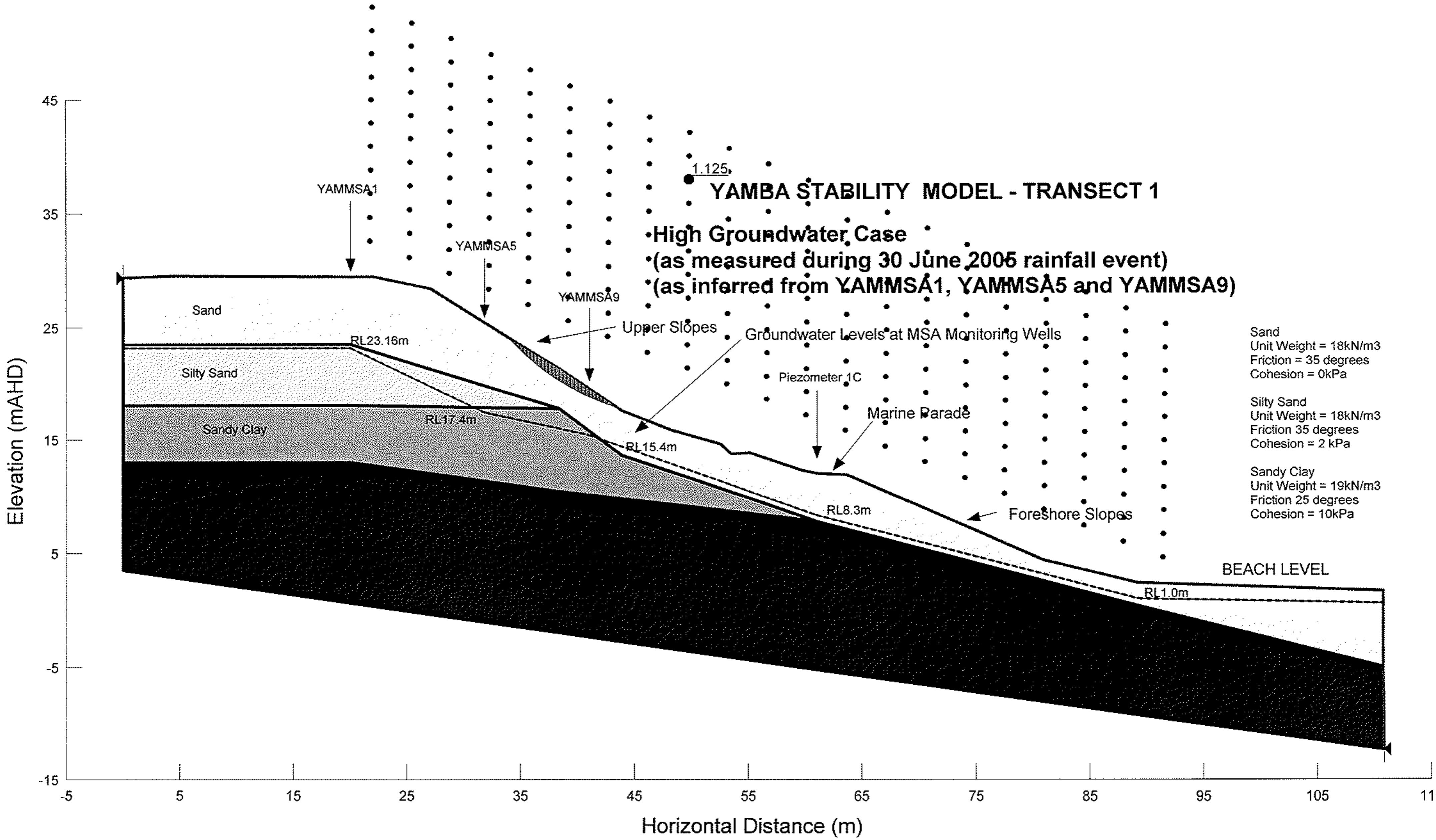
Report No. 19314WL2 Figure No. A17



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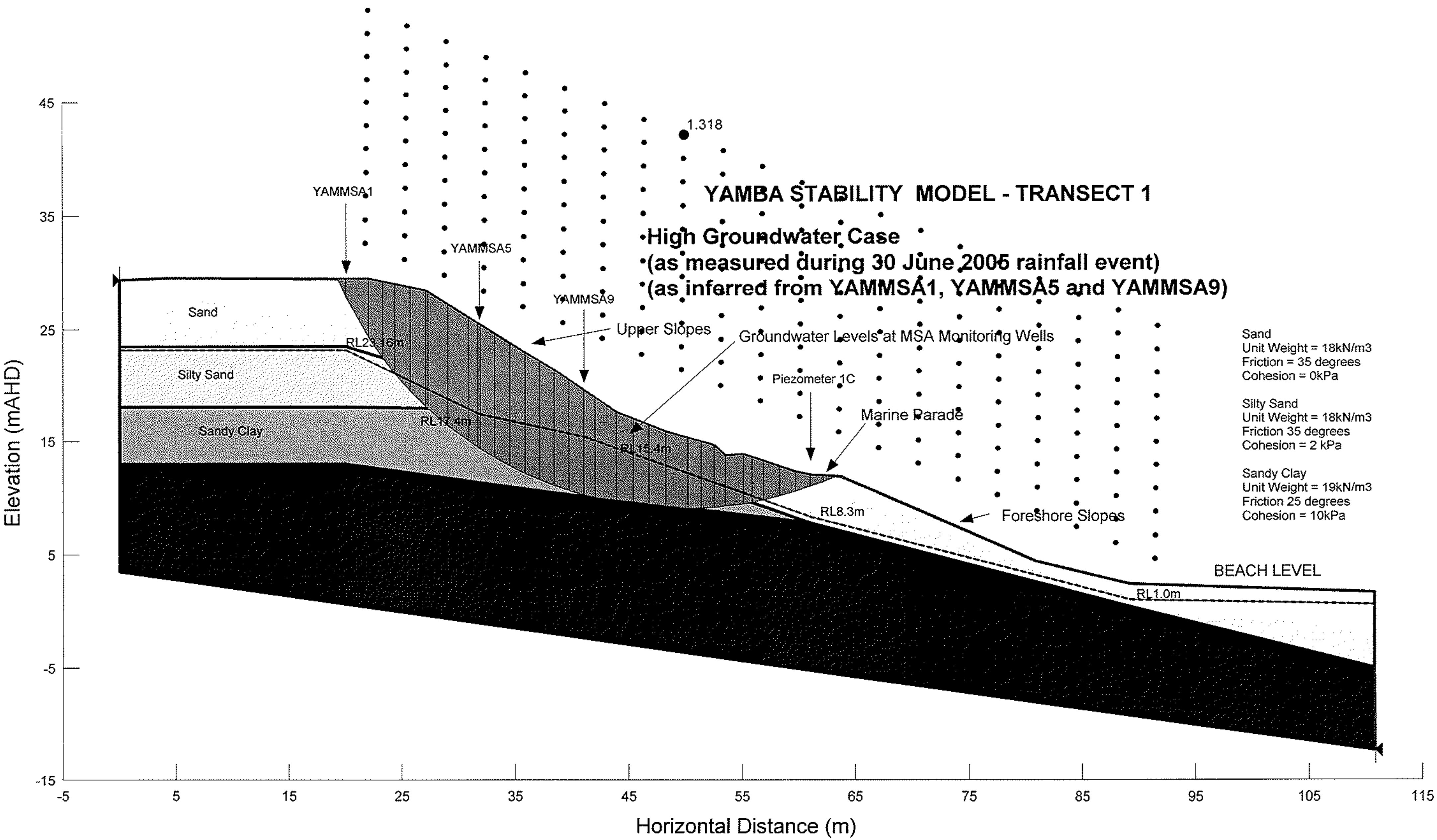
Report No. 19314WL2 Figure No. A18



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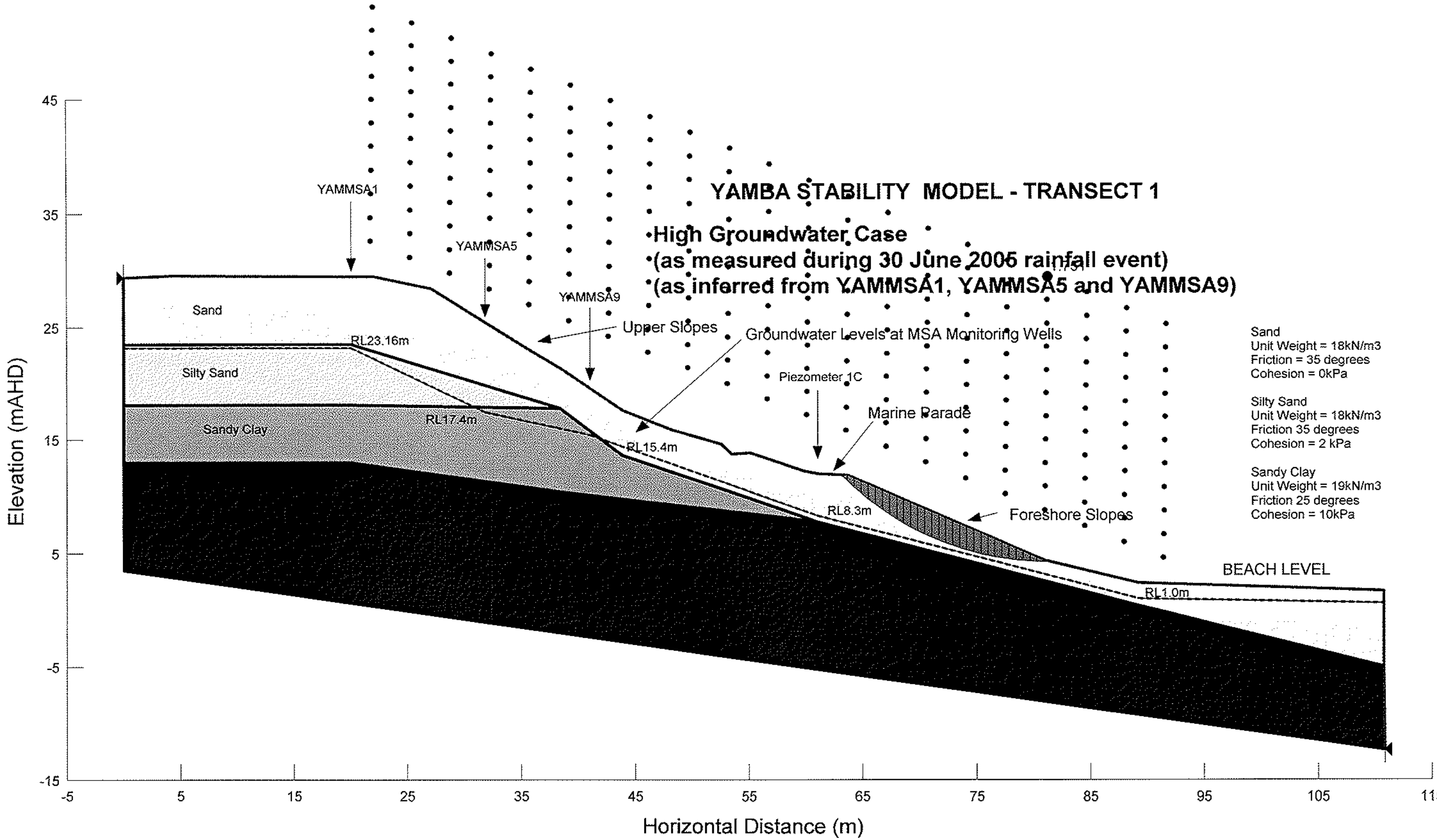
Report No. 19314WL2 Figure No. A19



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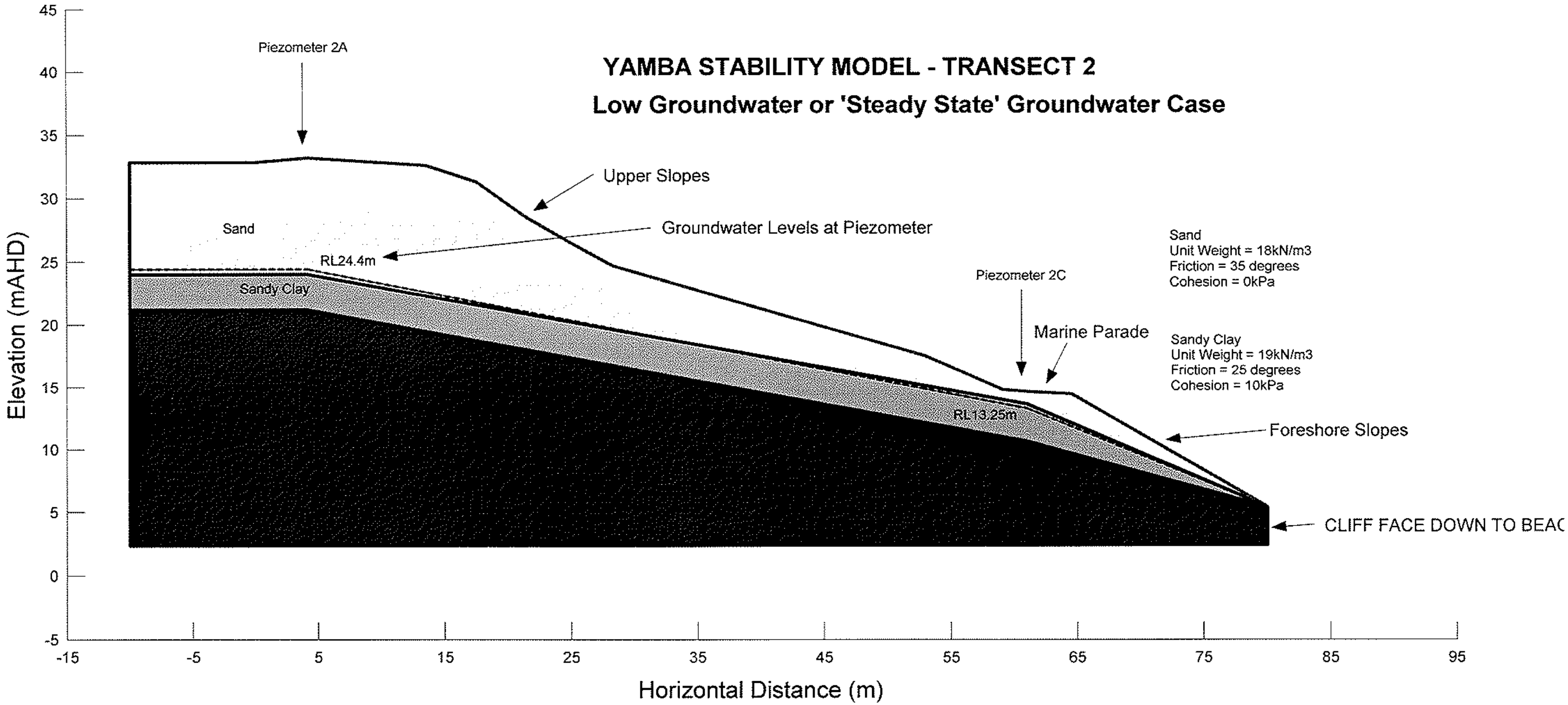
Report No. 19314WL2 Figure No. A20



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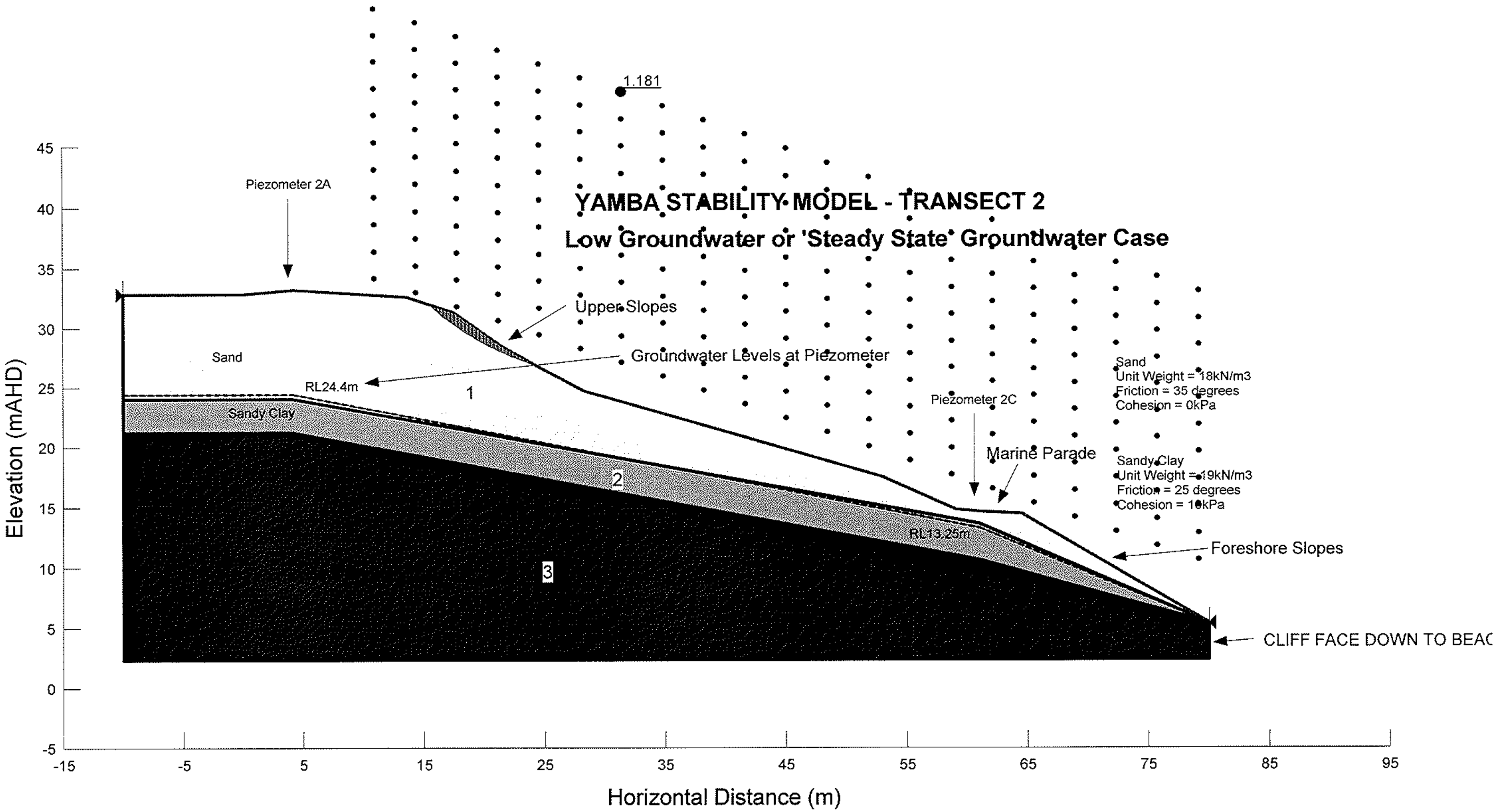
Report No. 19314WL2 Figure No. A21



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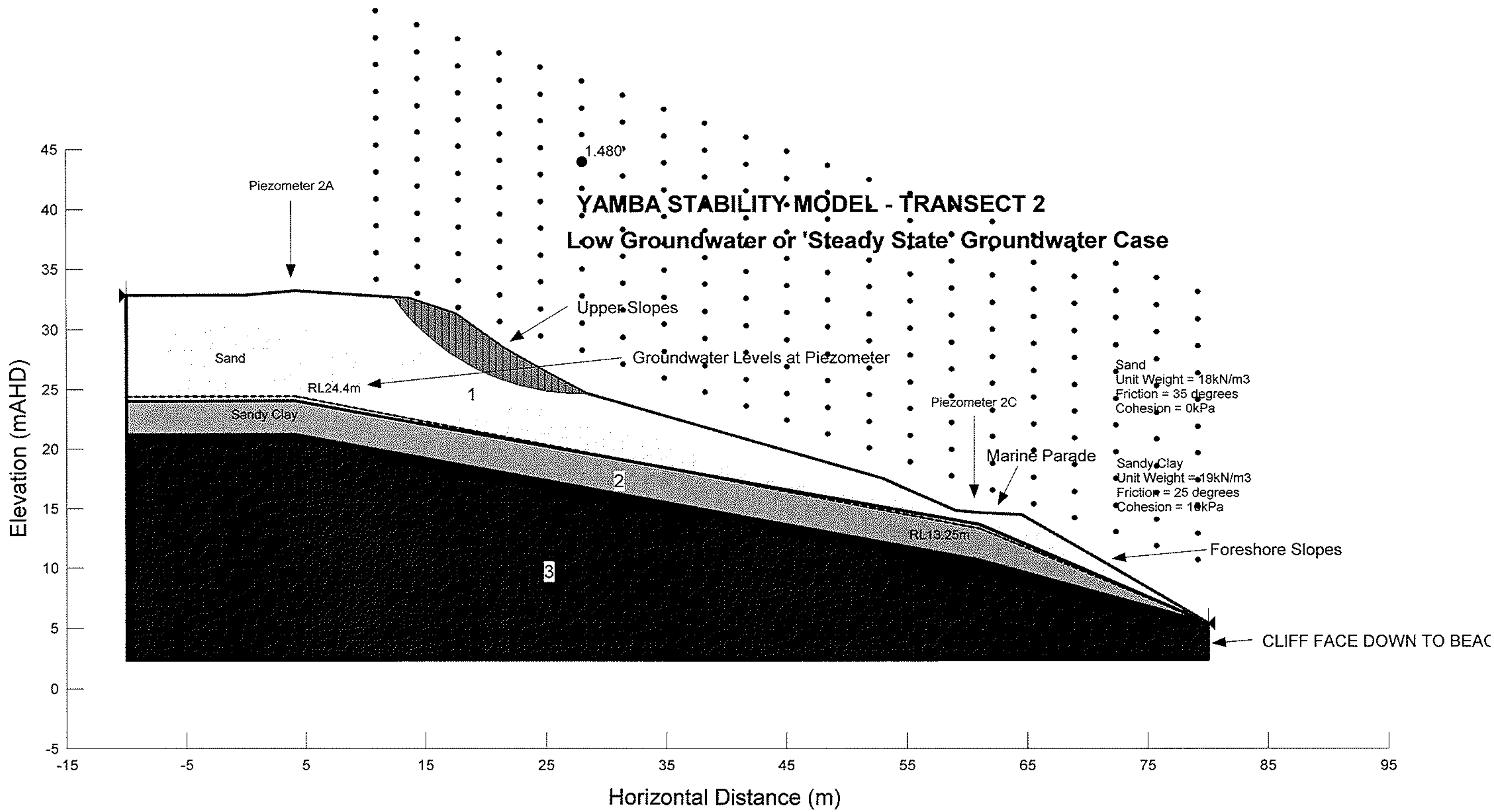

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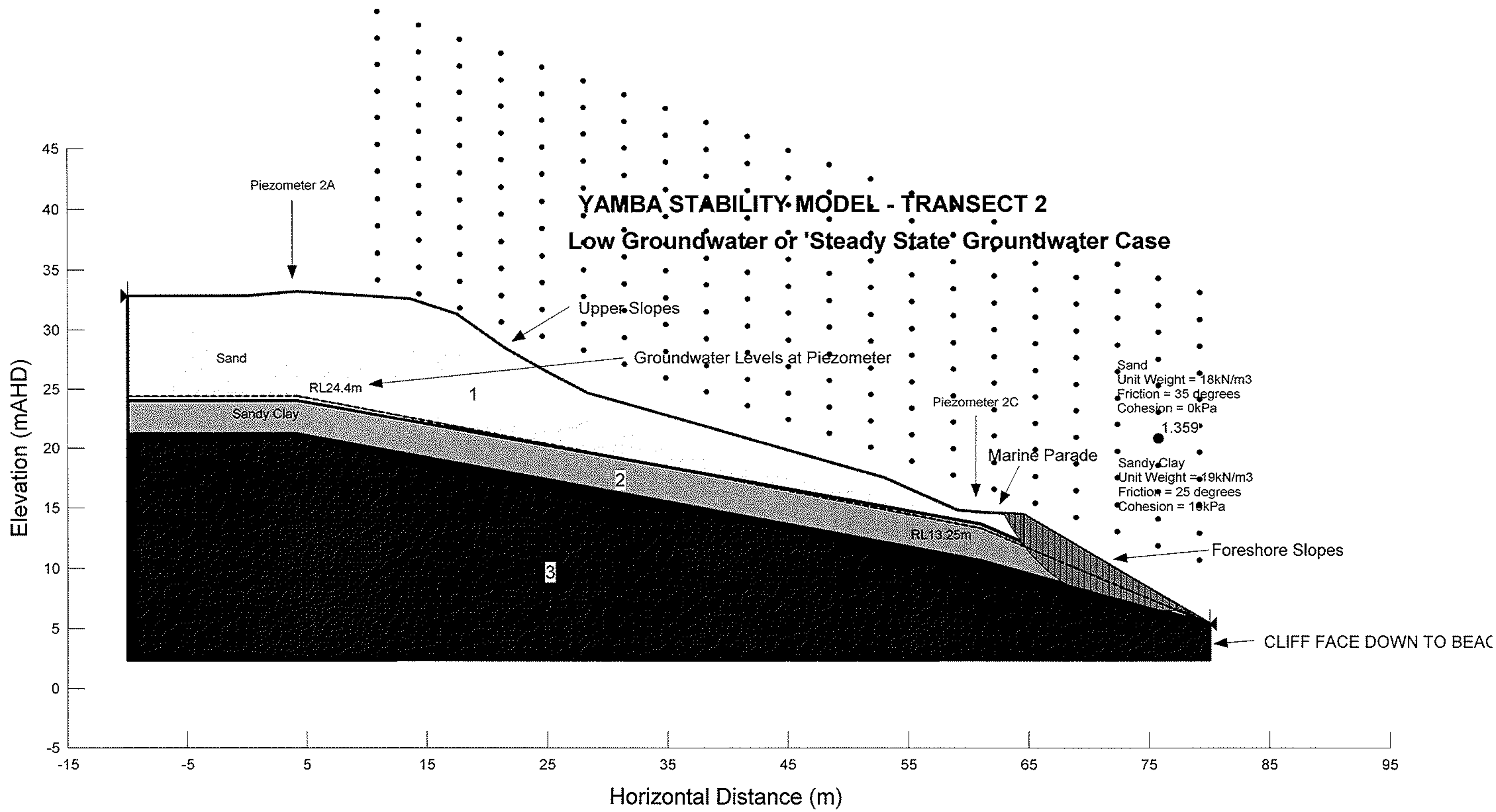
Report No. 19314WL2 Figure No. A23



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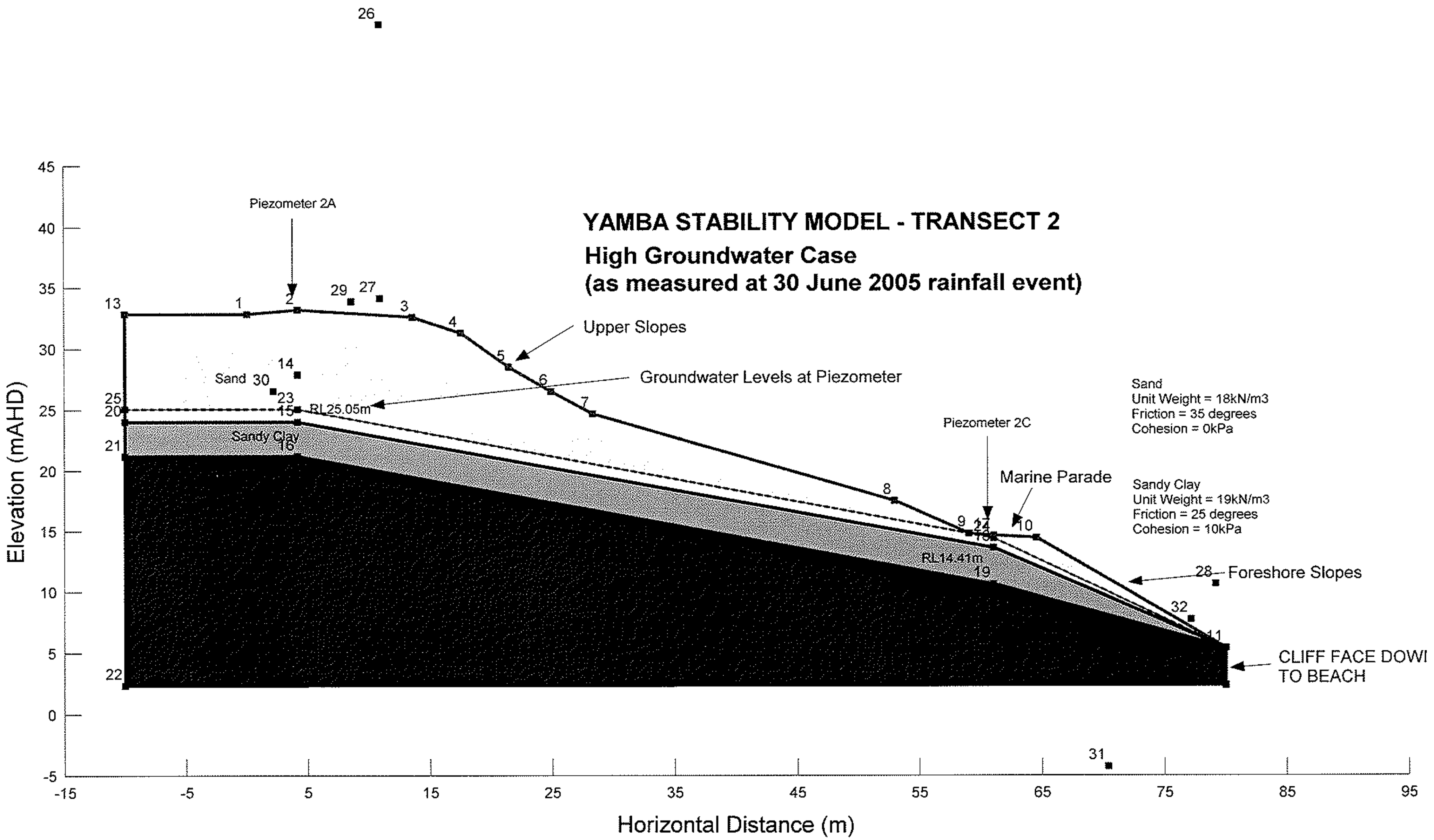
Report No. 19314WL2 Figure No. A24



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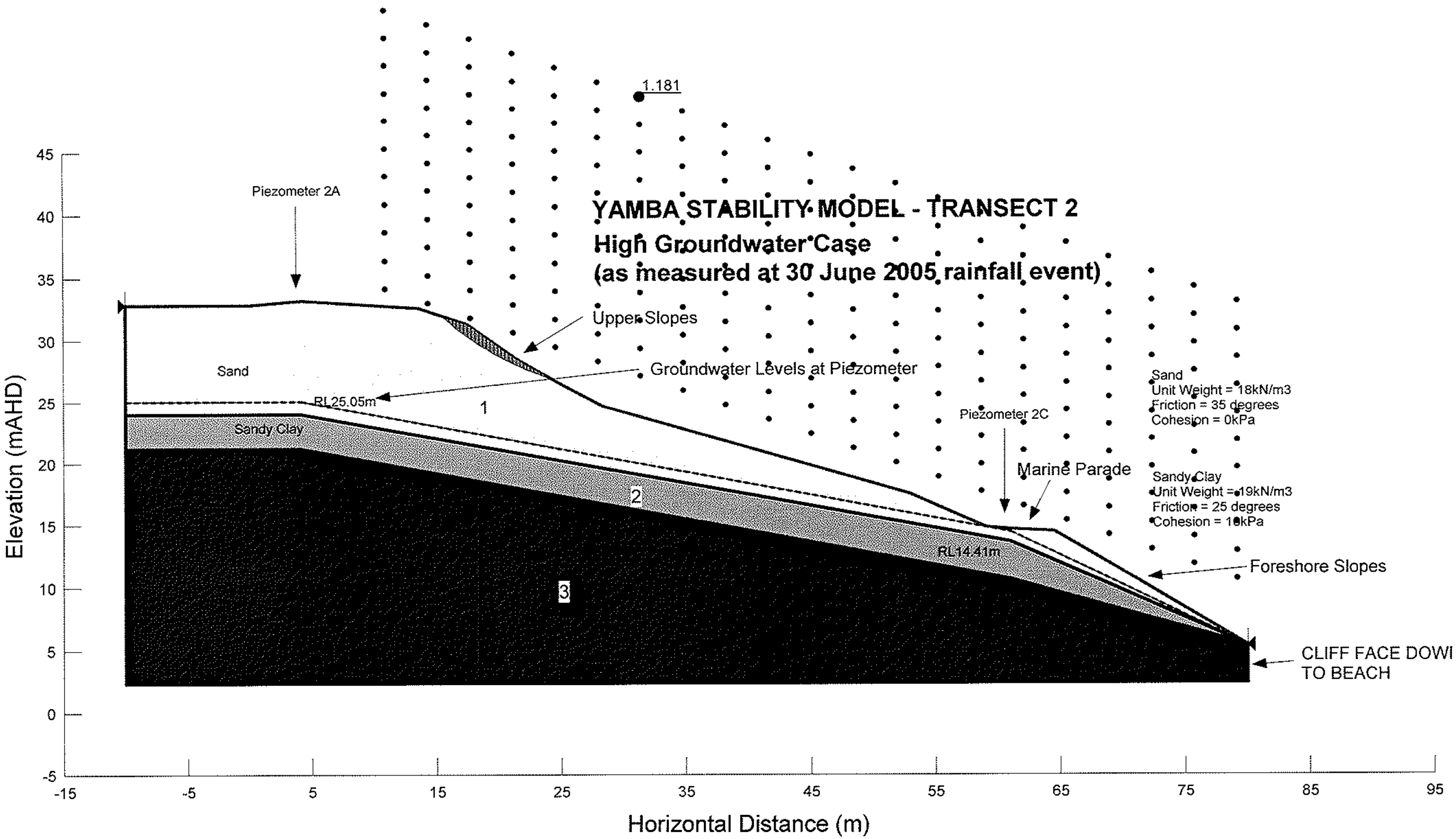
Report No. 19314WL2 Figure No. A25



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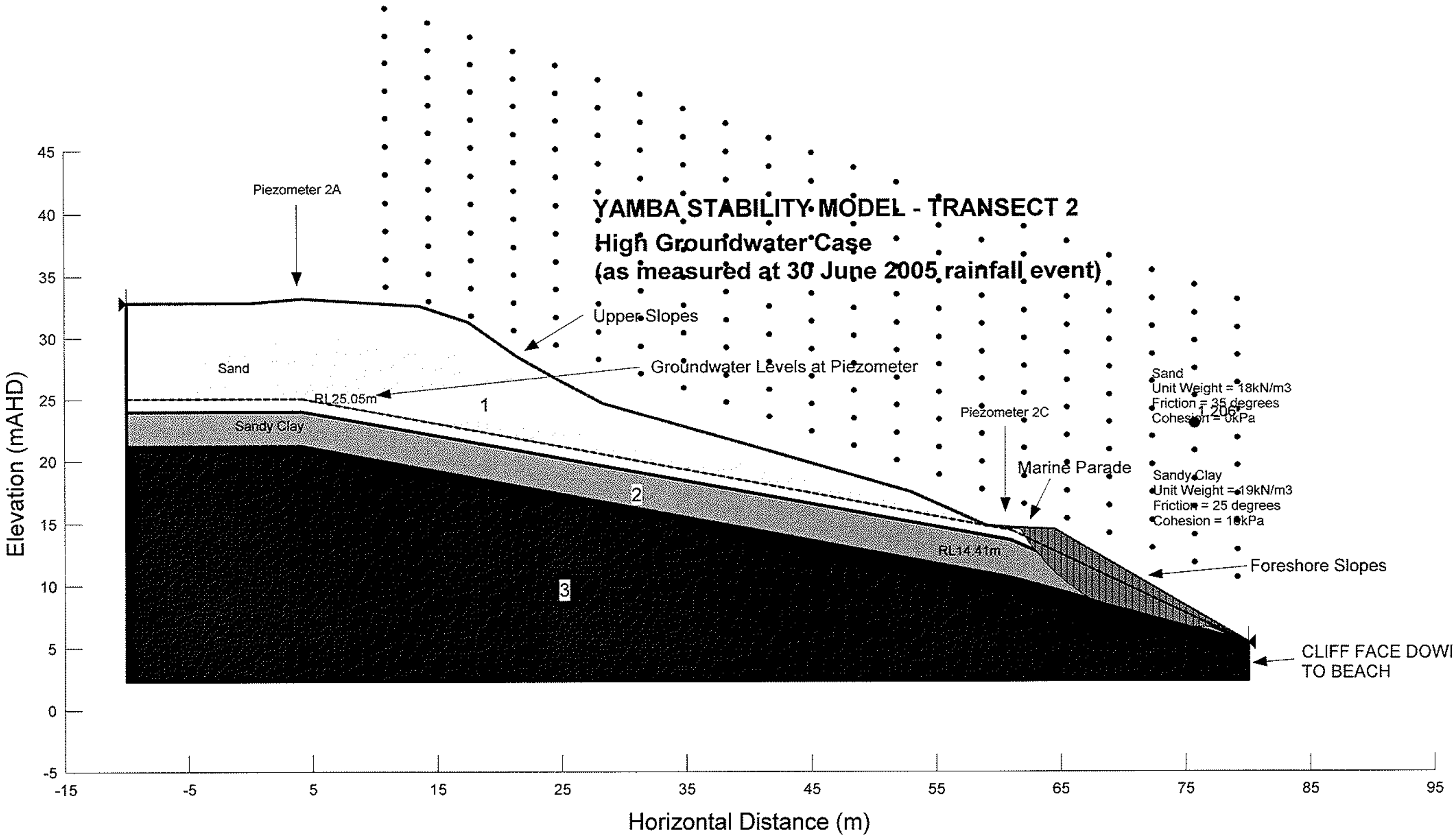
Report No. 19314WL2 Figure No. A26



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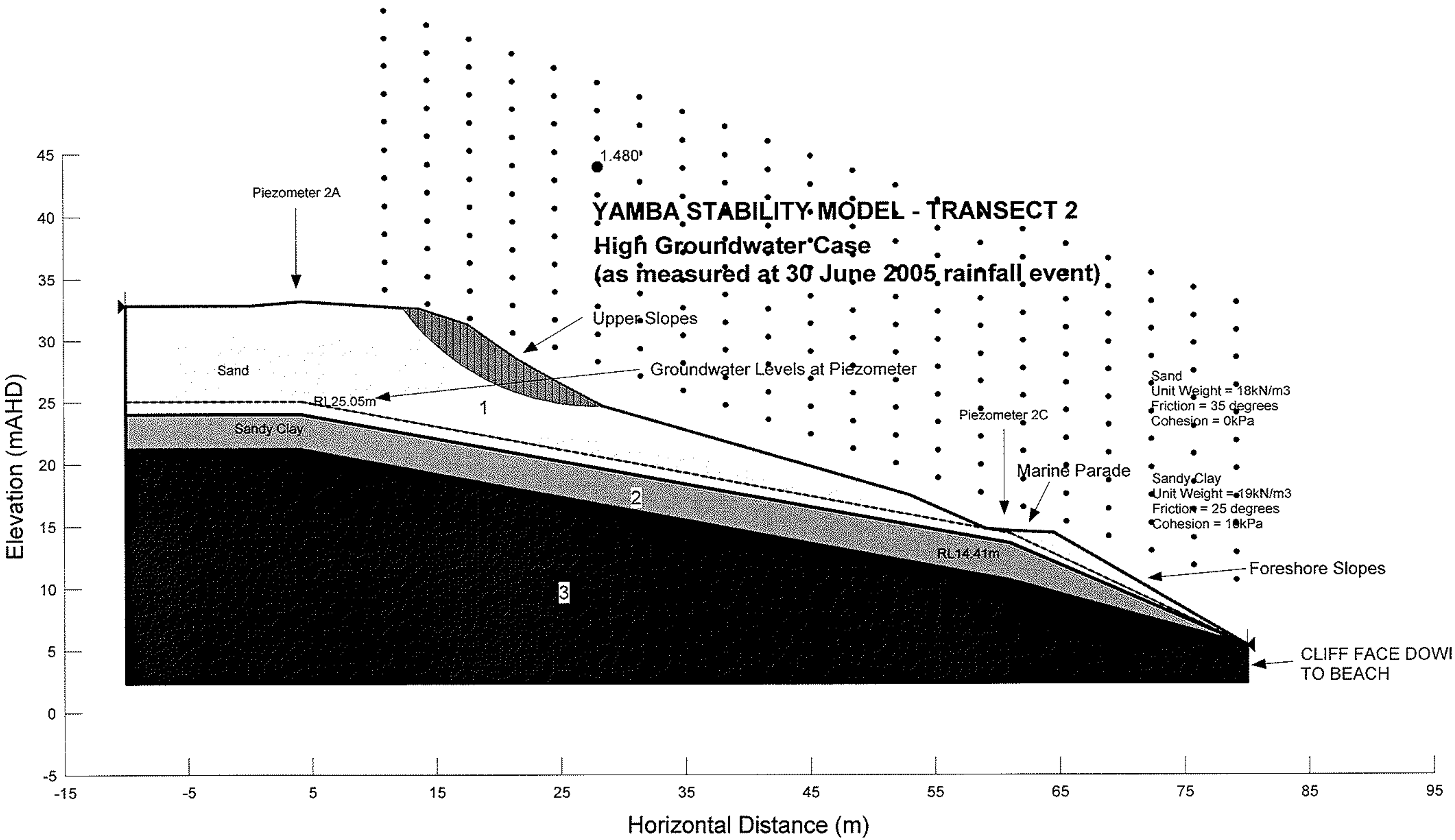
Report No. 19314WL2 Figure No. A27



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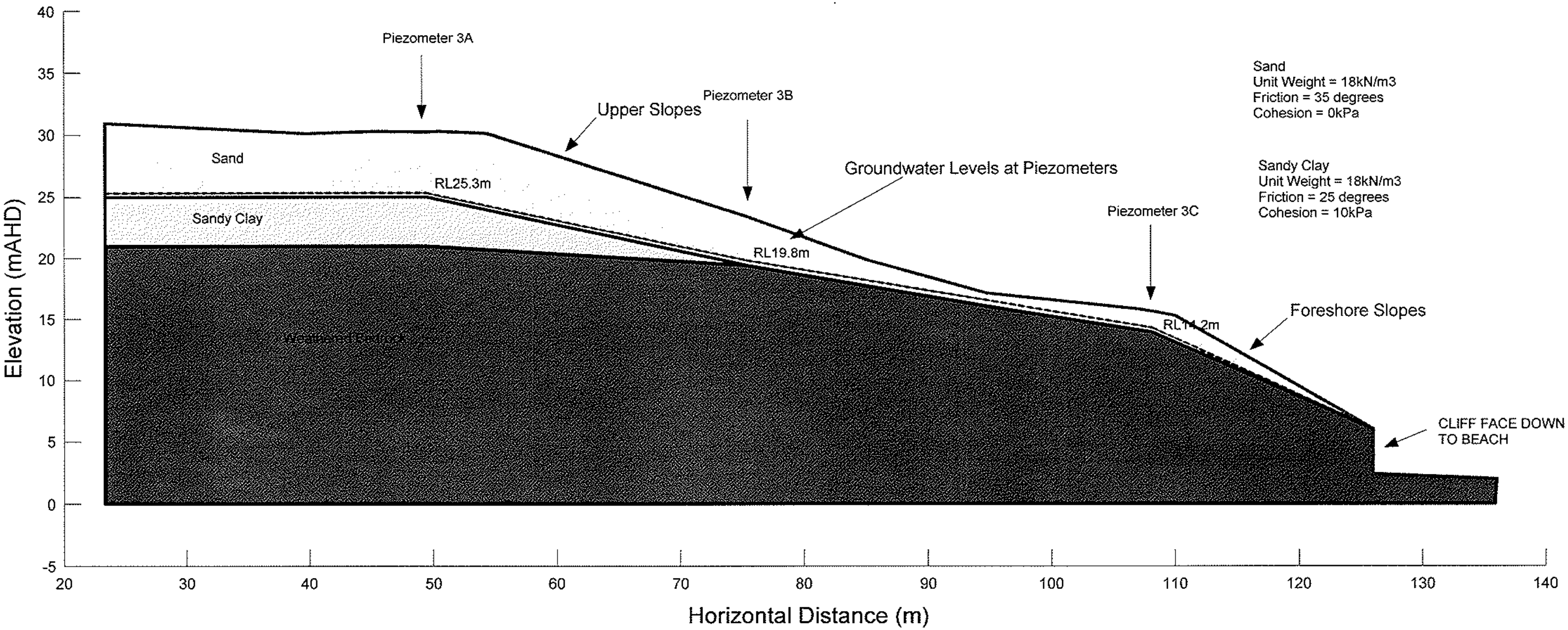
Report No. 19314WL2 Figure No. A28

YAMBA STABILITY MODEL - TRANSECT 3

Low Groundwater or 'Steady State' Groundwater Case



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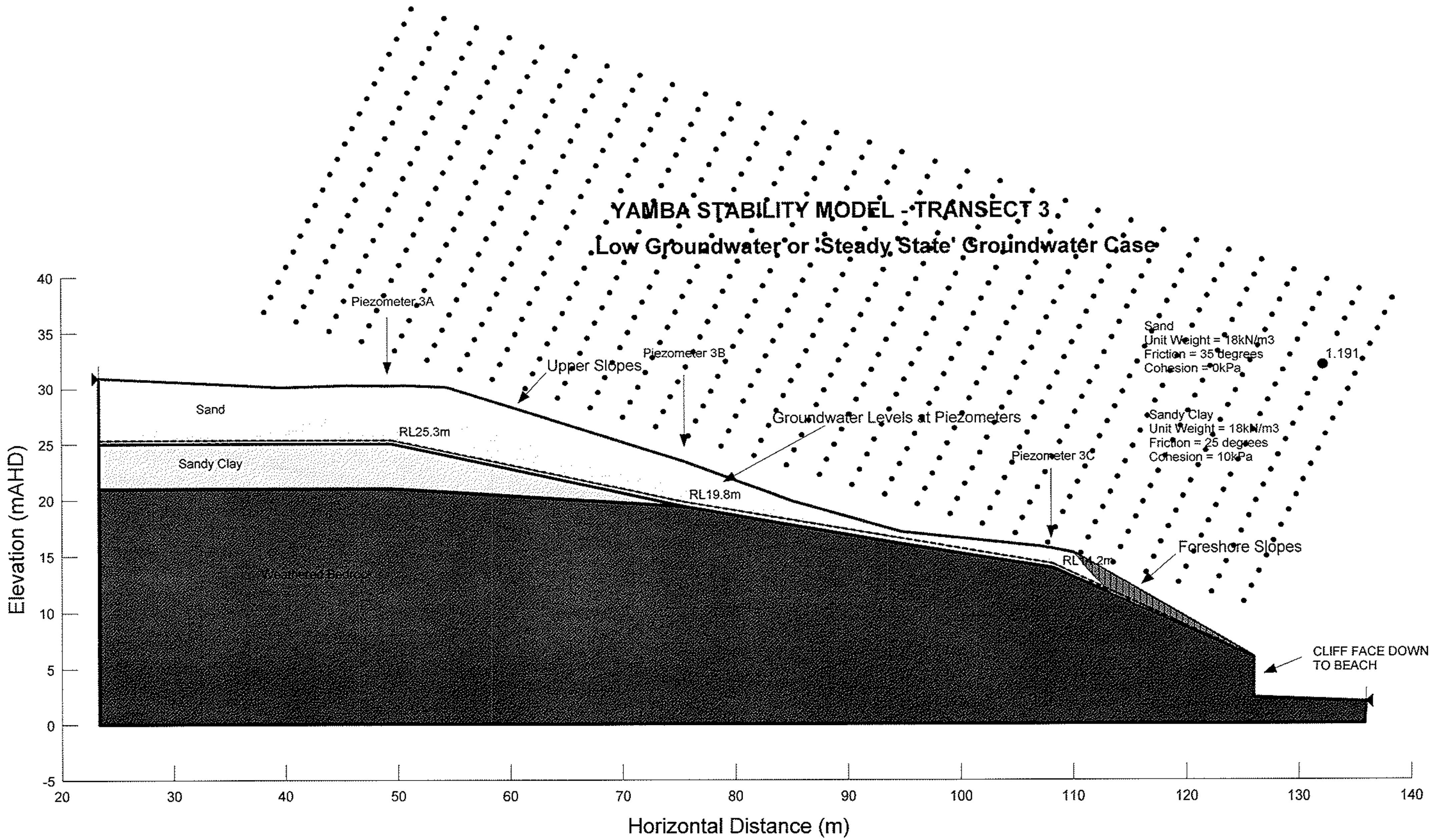
Report No. 19314WL2 Figure No. A29



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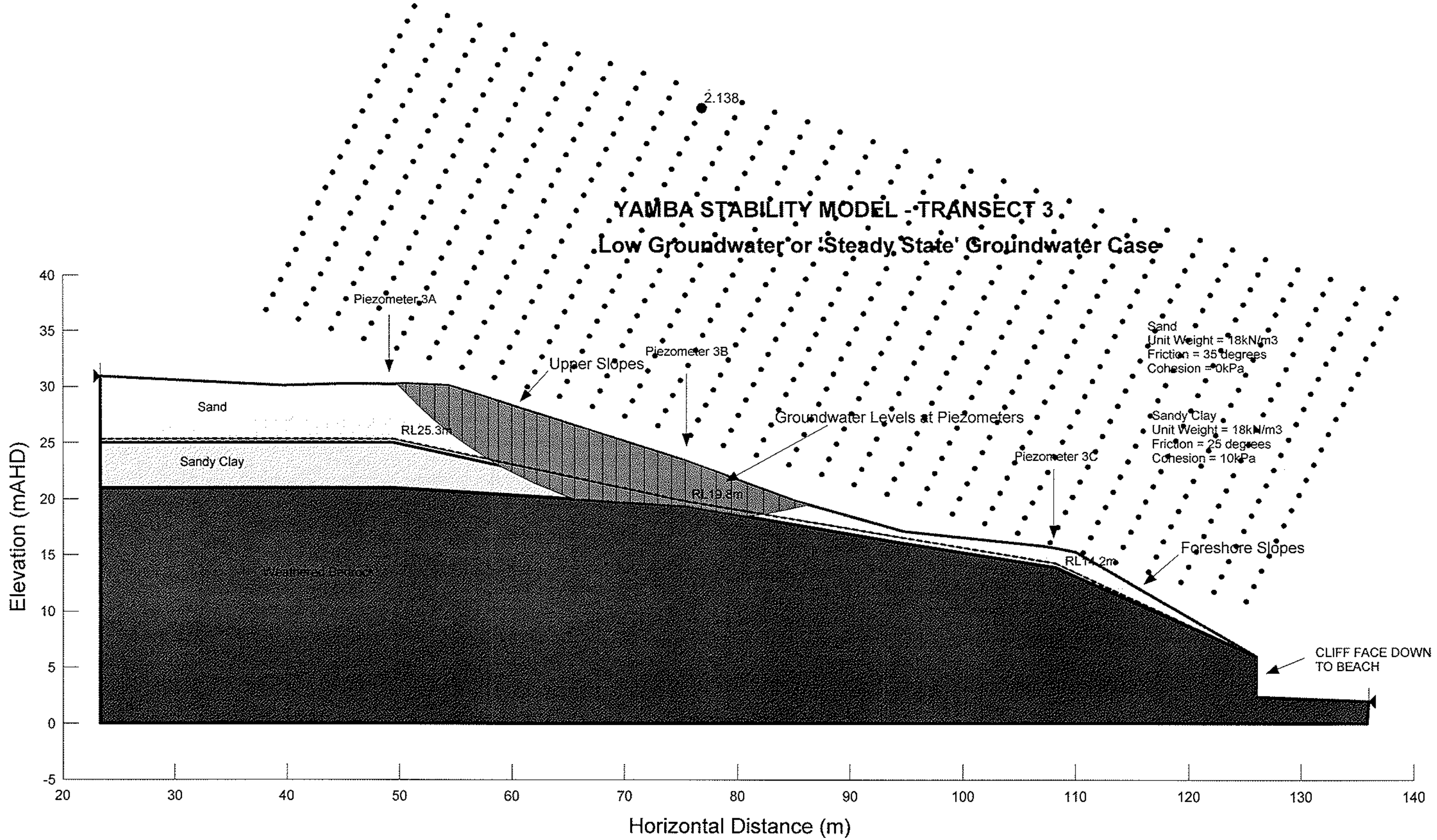
Report No. 19314WL2 Figure No. A30



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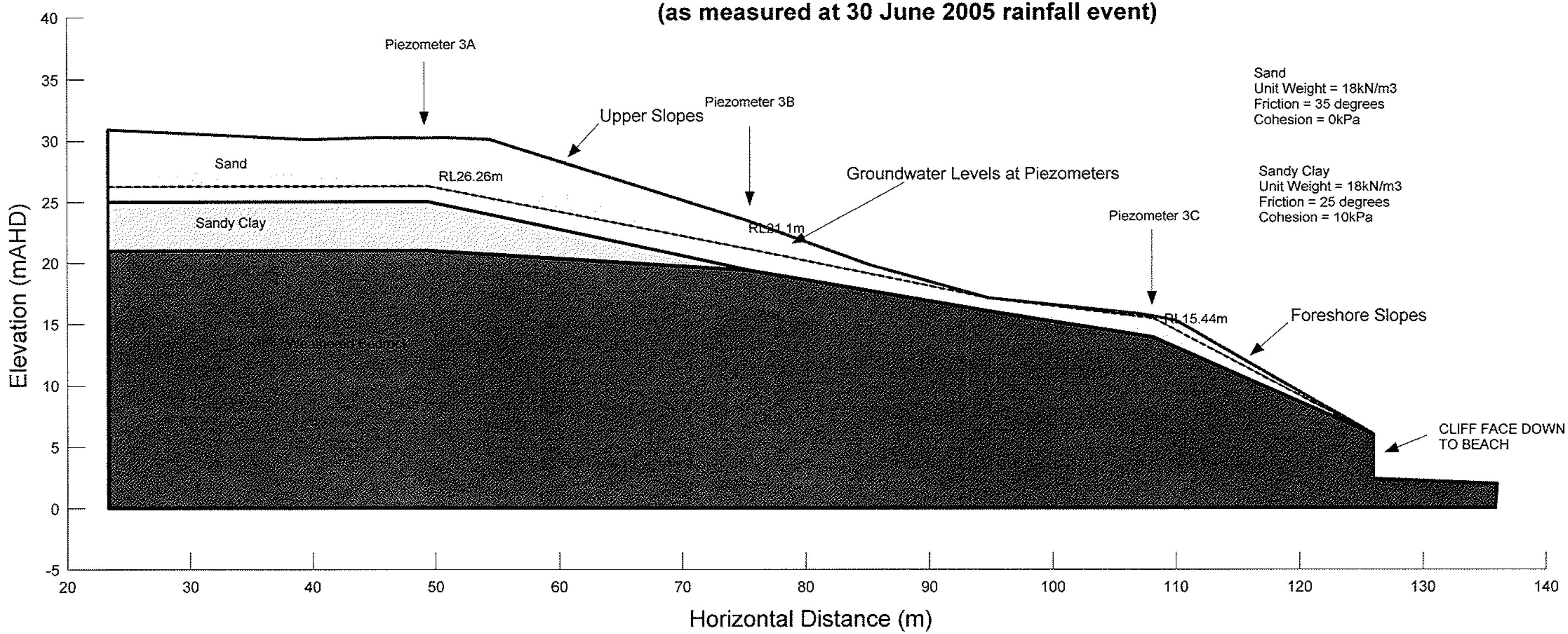
Report No. 19314WL2 Figure No. A31

YAMBA STABILITY MODEL - TRANSECT 3

High Groundwater Case
(as measured at 30 June 2005 rainfall event)



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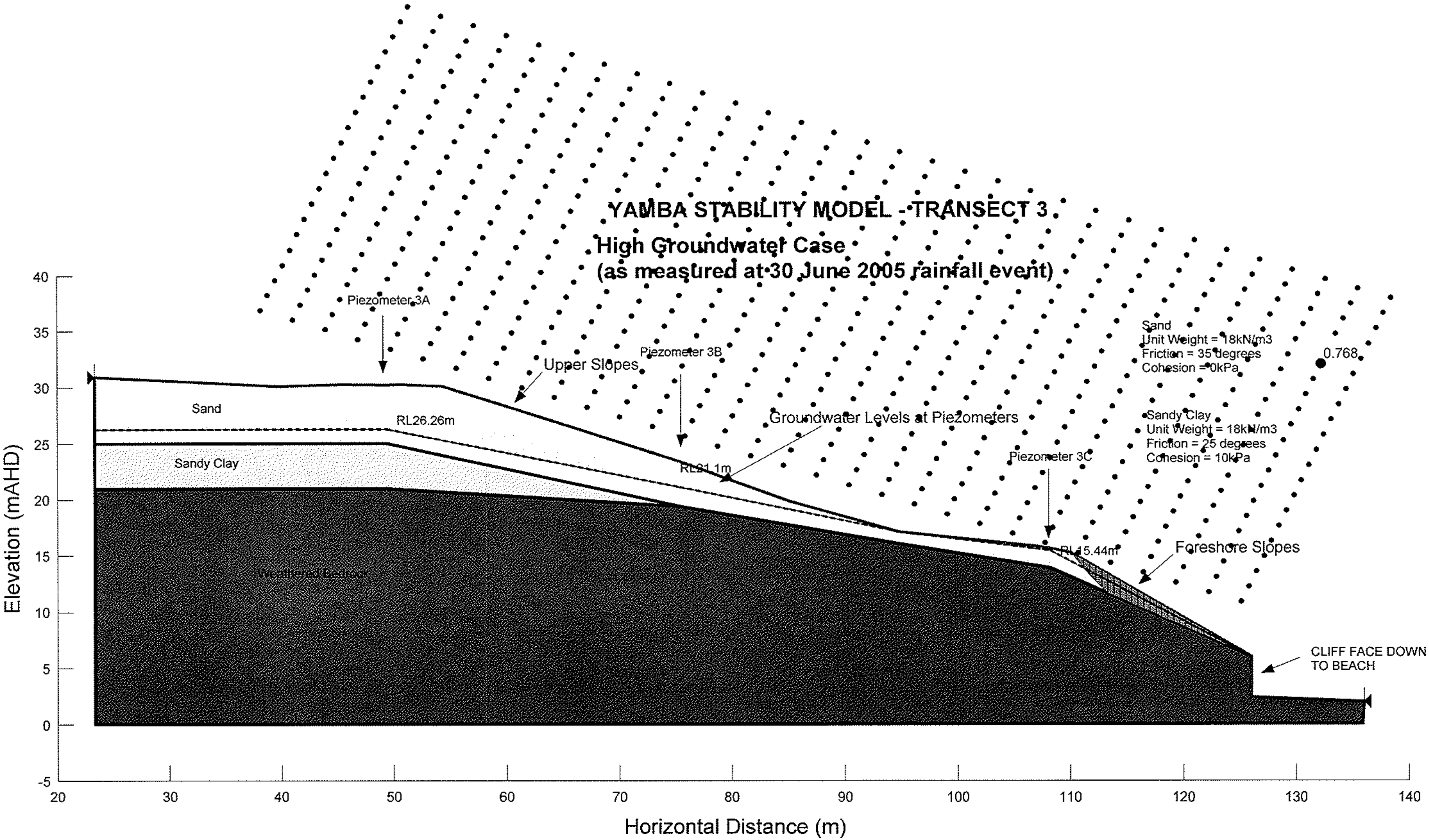
Report No. 19314WL2 Figure No. A32



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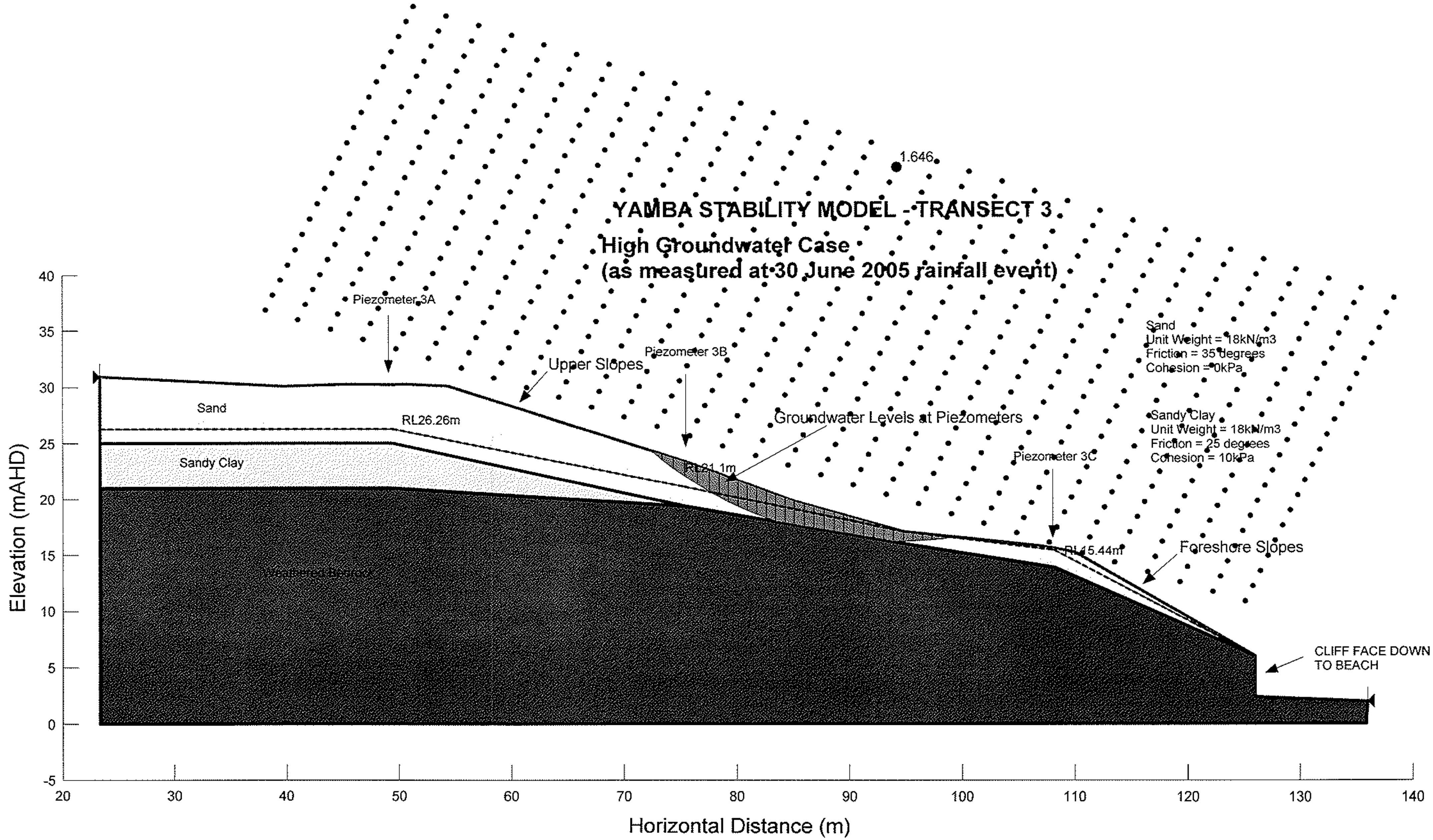
Report No. 19314WL2 Figure No. A33



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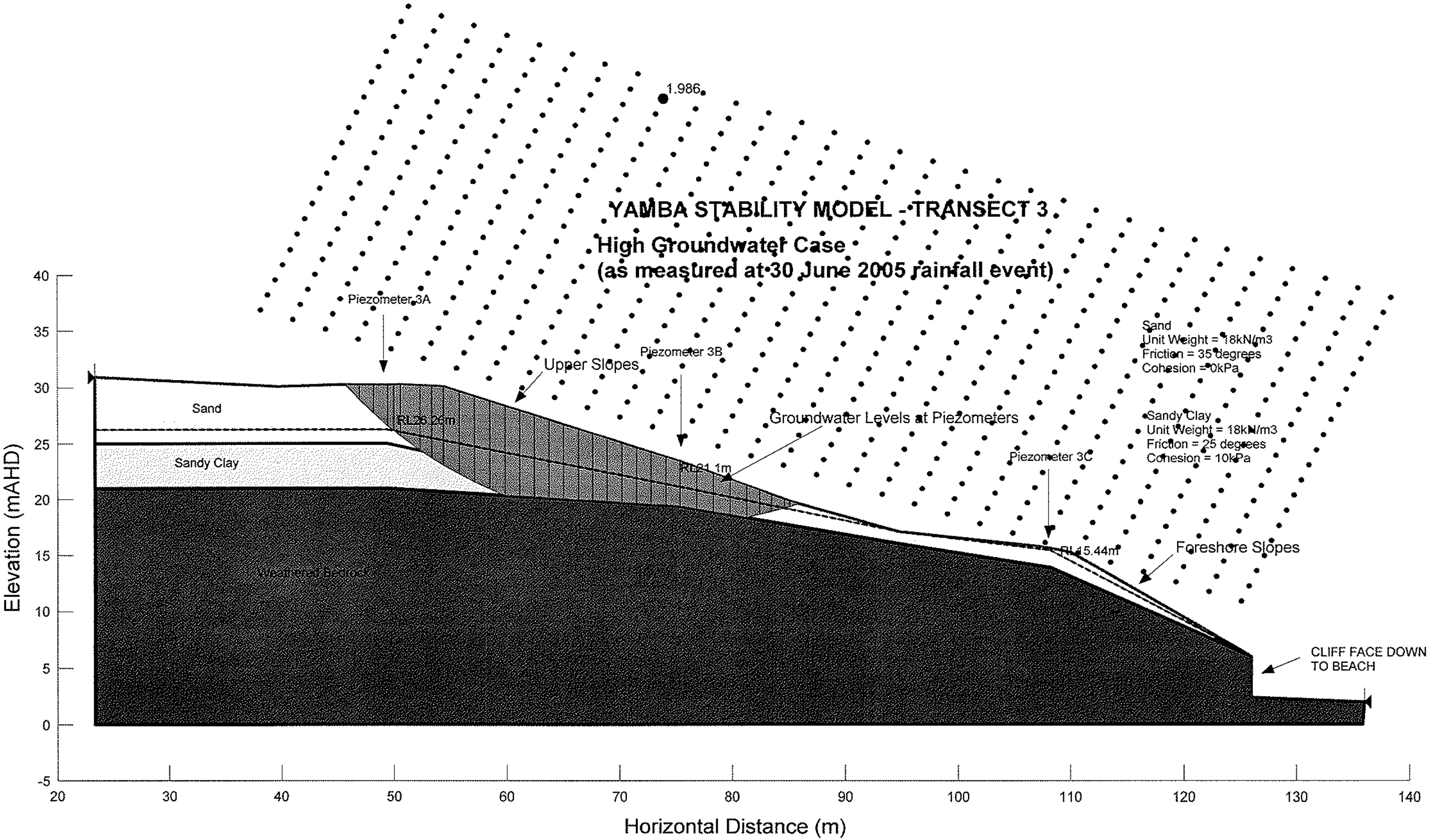
Report No. 19314WL2 Figure No. A34



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Report No. 19314WL2 Figure No. A35

APPENDIX B

TABLE B1 – Top 25 Ranked Rainfall Events in Monitoring Period

1 Day Rainfall

Rank	Record	Date	Rainfall
1	61	30/06/2005	250.4
2	486	30/08/2006	155.8
3	13	13/05/2005	94.4
4	207	24/11/2005	75.8
5	62	1/07/2005	73
6	557	9/11/2006	68.4
7	307	4/03/2006	66.4
8	253	9/01/2006	62.4
9	786	26/06/2007	60.8
10	264	20/01/2006	54.2
11	451	26/07/2006	48.9
12	230	17/12/2005	47.2
13	653	13/02/2007	46.6
14	600	22/12/2006	43.2
15	708	9/04/2007	40
16	767	7/06/2007	39.5
17	380	16/05/2006	38.6
18	677	9/03/2007	38
19	60	29/06/2005	37
20	12	12/05/2005	35.8
21	308	5/03/2006	34.4
22	417	22/06/2006	32.2
23	498	11/09/2006	30.4
24	416	21/06/2006	30
25	497	10/09/2006	29.4

2 Day Rainfall

Rank	Record	Date	Rainfall
1	62	1/07/2005	323.4
2	61	30/06/2005	287.4
3	486	30/08/2006	181.4
4	487	31/08/2006	169.2
5	13	13/05/2005	130.2
6	308	5/03/2006	100.8
7	14	14/05/2005	96.2
8	253	9/01/2006	80.2
9	208	25/11/2005	80.1
10	558	10/11/2006	77.6
11	786	26/06/2007	77.4
12	207	24/11/2005	76.4
13	557	9/11/2006	75
14	63	2/07/2005	73.4
15	254	10/01/2006	70.9
16	307	4/03/2006	70.9
17	767	7/06/2007	67.3
18	265	21/01/2006	63.1
19	417	22/06/2006	62.2
20	498	11/09/2006	59.8
21	264	20/01/2006	57.8
22	451	26/07/2006	53.9
23	416	21/06/2006	52
24	60	29/06/2005	50.8
25	212	29/11/2005	50.8

3 Day Rainfall

Rank	Record	Date	Rainfall
1	62	1/07/2005	360.4
2	63	2/07/2005	323.8
3	61	30/06/2005	301.2
4	487	31/08/2006	194.8
5	488	1/09/2006	186.4
6	486	30/08/2006	182.8
7	14	14/05/2005	132
8	13	13/05/2005	130.2
9	309	6/03/2006	116.4
10	308	5/03/2006	105.3
11	209	26/11/2005	99.6
12	15	15/05/2005	96.2
13	253	9/01/2006	88.8
14	254	10/01/2006	88.7
15	786	26/06/2007	87.4
16	417	22/06/2006	84.2
17	558	10/11/2006	84.2
18	208	25/11/2005	80.7
19	559	11/11/2006	77.6
20	207	24/11/2005	76.4
21	418	23/06/2006	76.4
22	557	9/11/2006	75.2
23	255	11/01/2006	73.5
24	64	3/07/2005	73.4
25	499	12/09/2006	73.4

NOTE: The date of the record represents the end of the period of antecedent rainfall

TABLE B1 (Continued) – Top 25 Ranked Rainfall Events in Monitoring Period

5 Day Rainfall

Rank	Record	Date	Rainfall
1	62	1/07/2005	376.3
2	63	2/07/2005	374.6
3	64	3/07/2005	360.8
4	65	4/07/2005	323.8
5	61	30/06/2005	309.3
6	488	1/09/2006	213.4
7	489	2/09/2006	213.2
8	487	31/08/2006	196.6
9	490	3/09/2006	188.2
10	486	30/08/2006	183.2
11	16	16/05/2005	132.4
12	14	14/05/2005	132
13	15	15/05/2005	132
14	308	5/03/2006	130.8
15	13	13/05/2005	130.2
16	310	7/03/2006	129.3
17	211	28/11/2005	126.6
18	311	8/03/2006	125
19	309	6/03/2006	121.5
20	254	10/01/2006	118.9
21	419	24/06/2006	111.8
22	253	9/01/2006	110.9
23	418	23/06/2006	109.4
24	307	4/03/2006	102.4
25	209	26/11/2005	100.2

10 Day Rainfall

Rank	Record	Date	Rainfall
1	66	5/07/2005	385.7
2	63	2/07/2005	385.1
3	64	3/07/2005	385.1
4	65	4/07/2005	385.1
5	62	1/07/2005	384.7
6	67	6/07/2005	379.7
7	68	7/07/2005	377.6
8	69	8/07/2005	363.8
9	70	9/07/2005	326.8
10	61	30/06/2005	311.7
11	490	3/09/2006	215.8
12	491	4/09/2006	215.6
13	492	5/09/2006	215.6
14	489	2/09/2006	215.2
15	493	6/09/2006	215.2
16	494	7/09/2006	214.2
17	488	1/09/2006	214
18	487	31/08/2006	197
19	495	8/09/2006	188.6
20	486	30/08/2006	183.6
21	216	3/12/2005	173.4
22	311	8/03/2006	162.4
23	310	7/03/2006	162.2
24	312	9/03/2006	161
25	215	2/12/2005	160.8

15 Day Rainfall

Rank	Record	Date	Rainfall
1	63	2/07/2005	388.4
2	64	3/07/2005	388.4
3	66	5/07/2005	388.1
4	67	6/07/2005	388.1
5	68	7/07/2005	388.1
6	69	8/07/2005	388.1
7	70	9/07/2005	388.1
8	62	1/07/2005	388
9	65	4/07/2005	386.1
10	71	10/07/2005	385.7
11	72	11/07/2005	379.7
12	73	12/07/2005	377.6
13	74	13/07/2005	363.8
14	75	14/07/2005	328
15	61	30/06/2005	315
16	499	12/09/2006	287.6
17	498	11/09/2006	275.4
18	500	13/09/2006	265.4
19	497	10/09/2006	245.4
20	490	3/09/2006	216.3
21	494	7/09/2006	216.2
22	495	8/09/2006	216.2
23	491	4/09/2006	216
24	492	5/09/2006	216
25	496	9/09/2006	216

TABLE B1 (Continued) – Top 25 Ranked Rainfall Events in Monitoring Period

30 Day Rainfall

Rank	Record	Date	Rainfall
1	85	24/07/2005	405.5
2	66	5/07/2005	405.3
3	67	6/07/2005	405.3
4	68	7/07/2005	405.1
5	84	23/07/2005	405.1
6	83	22/07/2005	404.7
7	69	8/07/2005	403.8
8	70	9/07/2005	403.2
9	71	10/07/2005	403.2
10	86	25/07/2005	403.1
11	62	1/07/2005	402.9
12	63	2/07/2005	402.9
13	75	14/07/2005	402.9
14	72	11/07/2005	402.8
15	64	3/07/2005	402.7
16	73	12/07/2005	402.5
17	65	4/07/2005	402.3
18	74	13/07/2005	401.7
19	87	26/07/2005	397.1
20	88	27/07/2005	395
21	76	15/07/2005	393.6
22	77	16/07/2005	393.6
23	78	17/07/2005	393.6
24	79	18/07/2005	393.6
25	82	21/07/2005	391.7

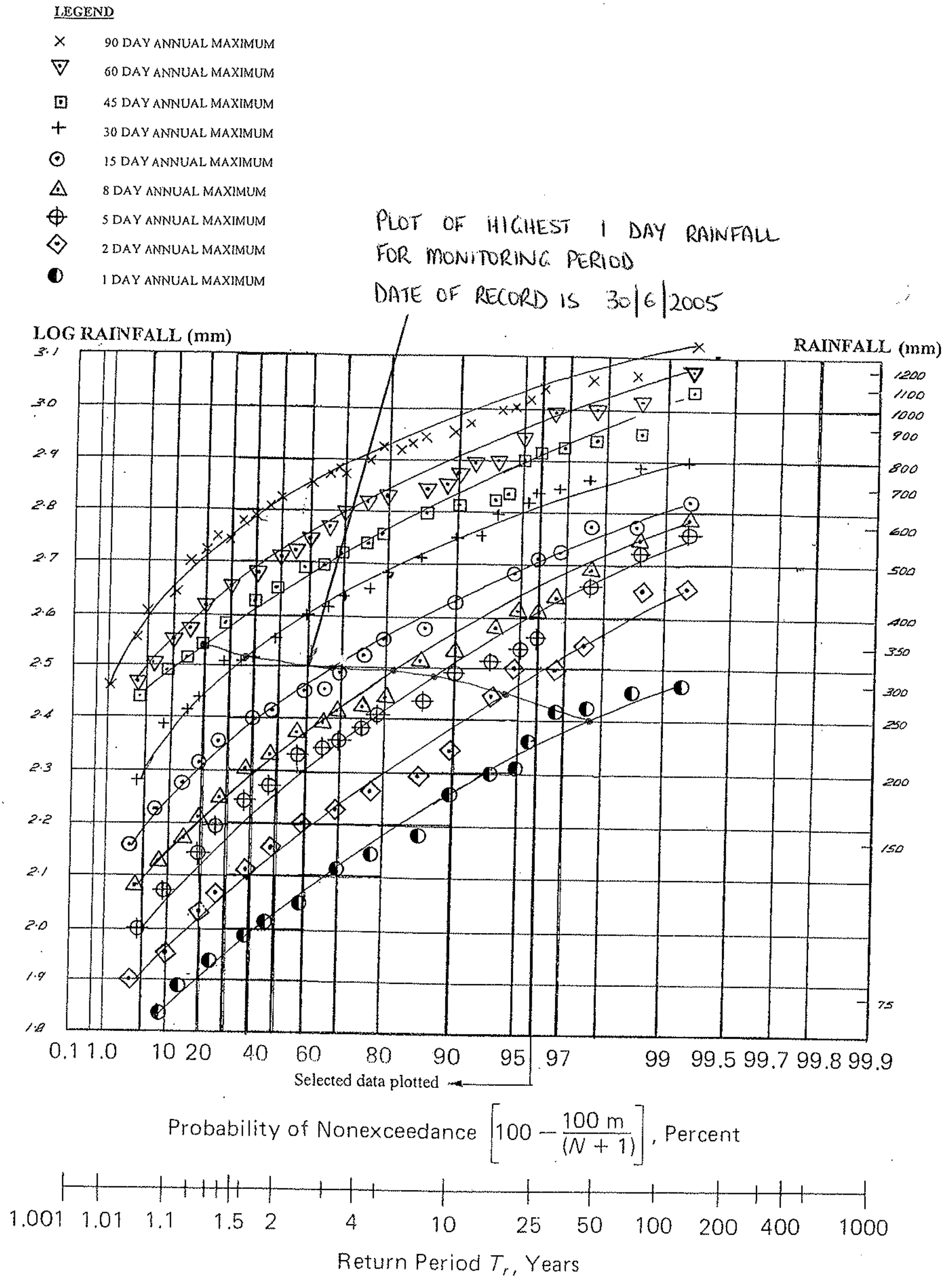
45 Day Rainfall

Rank	Record	Date	Rainfall
1	62	1/07/2005	426.1
2	63	2/07/2005	425.5
3	64	3/07/2005	425.5
4	65	4/07/2005	423.3
5	83	22/07/2005	421.7
6	84	23/07/2005	420.8
7	85	24/07/2005	420.6
8	86	25/07/2005	420.6
9	87	26/07/2005	420.2
10	88	27/07/2005	419.9
11	90	29/07/2005	419.3
12	89	28/07/2005	419.1
13	97	5/08/2005	411.4
14	98	6/08/2005	411.4
15	99	7/08/2005	411.4
16	100	8/08/2005	411.4
17	94	2/08/2005	410
18	92	31/07/2005	409.8
19	93	1/08/2005	409.8
20	91	30/07/2005	409.4
21	101	9/08/2005	409
22	82	21/07/2005	408.9
23	76	15/07/2005	408.5
24	77	16/07/2005	408.5
25	81	20/07/2005	408.5

90 Day Rainfall

Rank	Record	Date	Rainfall
1	91	30/07/2005	600.2
2	90	29/07/2005	600
3	92	31/07/2005	593.4
4	93	1/08/2005	593.4
5	94	2/08/2005	591
6	95	3/08/2005	588.4
7	97	5/08/2005	588.4
8	96	4/08/2005	585.6
9	98	6/08/2005	585.2
10	99	7/08/2005	585.2
11	100	8/08/2005	585.2
12	101	9/08/2005	585.2
13	498	11/09/2006	581
14	502	15/09/2006	578.4
15	501	14/09/2006	577
16	500	13/09/2006	576.6
17	499	12/09/2006	573.8
18	503	16/09/2006	573.6
19	504	17/09/2006	563
20	490	3/09/2006	561.8
21	491	4/09/2006	561.8
22	290	15/02/2006	561.6
23	489	2/09/2006	561.2
24	492	5/09/2006	561.2
25	488	1/09/2006	560

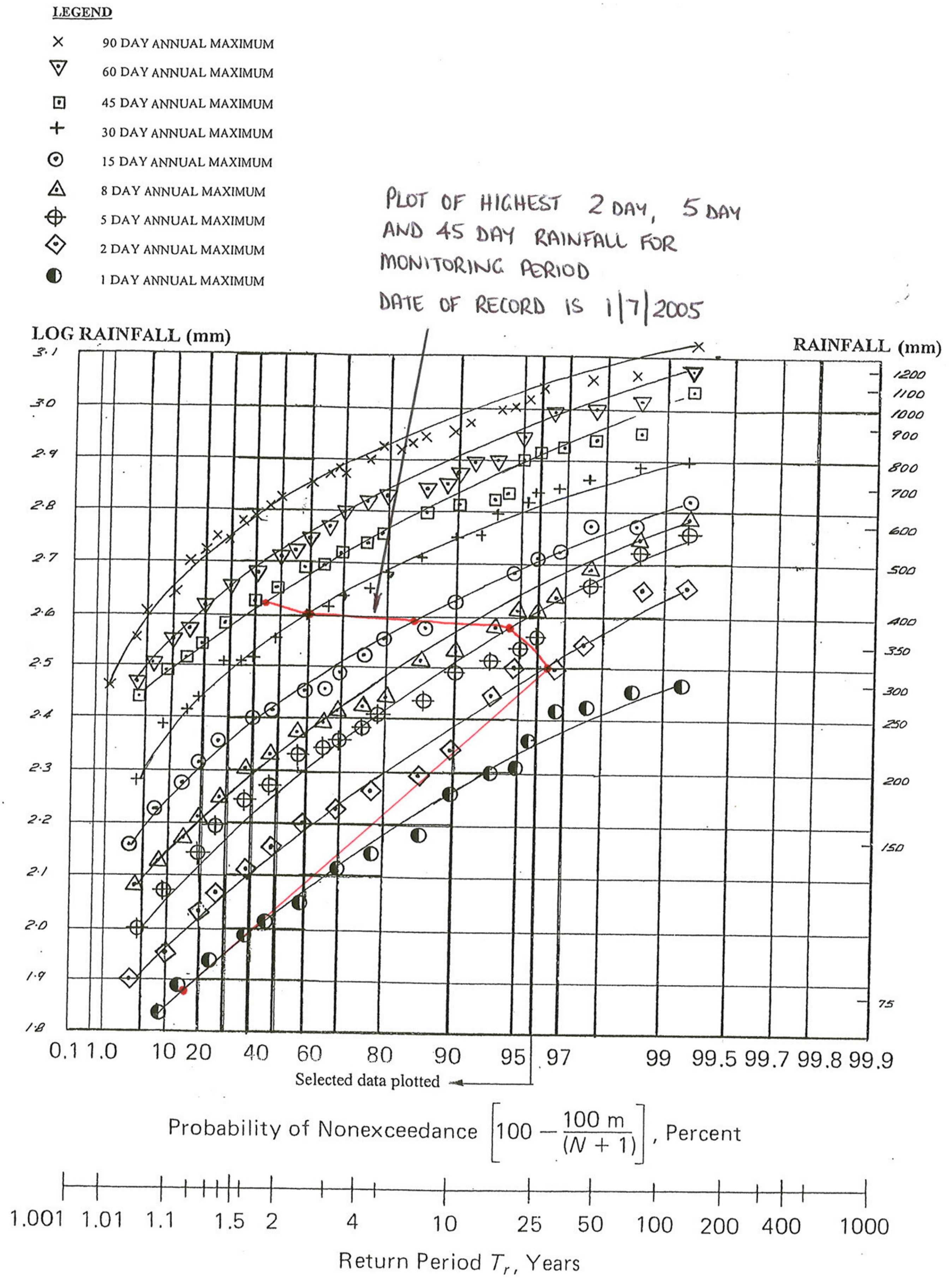
NOTE: The date of the record represents the end of the period of antecedent rainfall



PROBABILITY PLOT OF ACTUAL RAINFALL
AND ANTECEDANT RAINFALL FOR YAMBA

Jeffery & Katauskas Pty Ltd

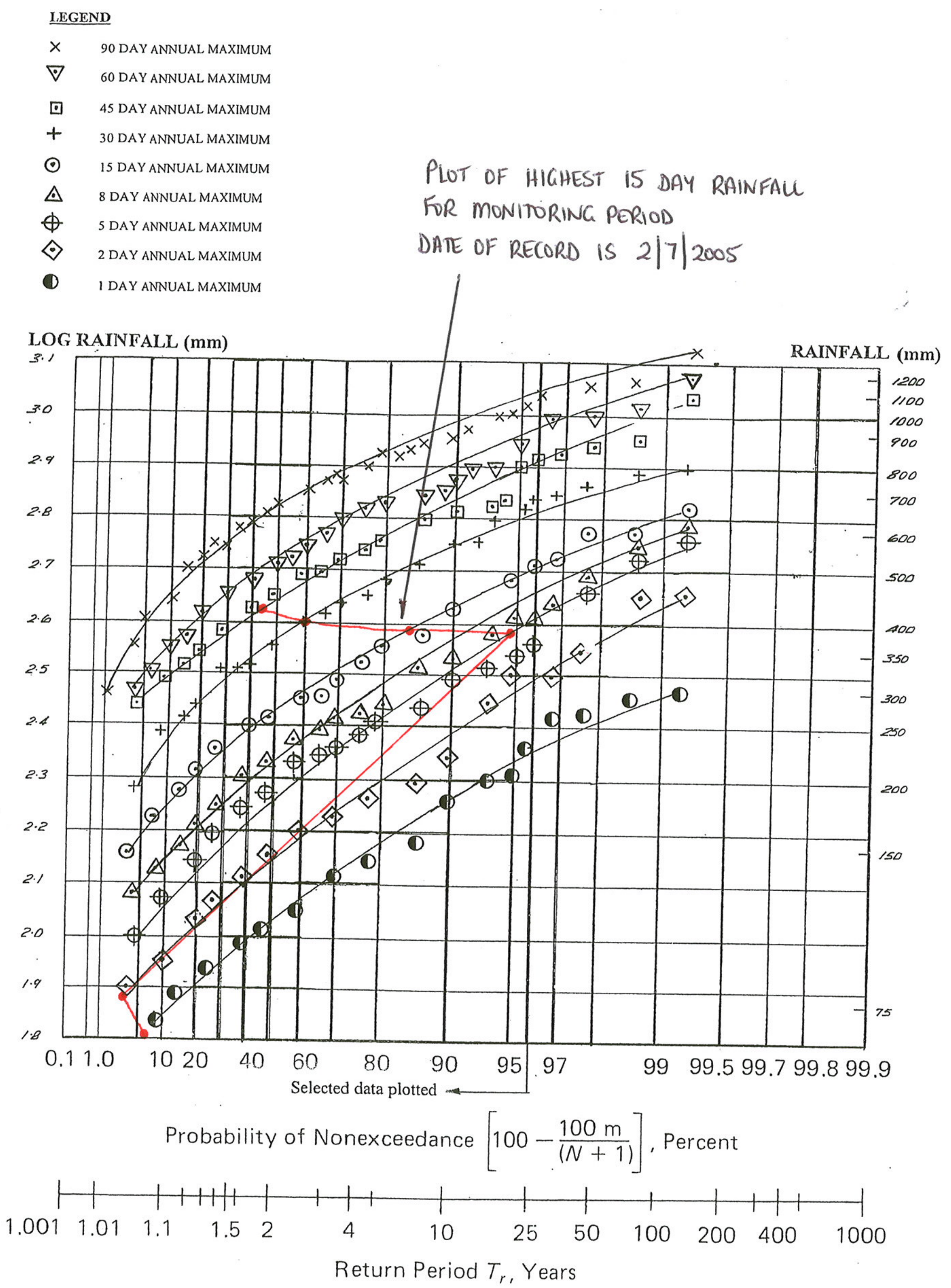
Report No. 19314WL Figure No. B1



PROBABILITY PLOT OF ACTUAL RAINFALL AND ANTECEDANT RAINFALL FOR YAMBA

Jeffery & Katauskas Pty Ltd

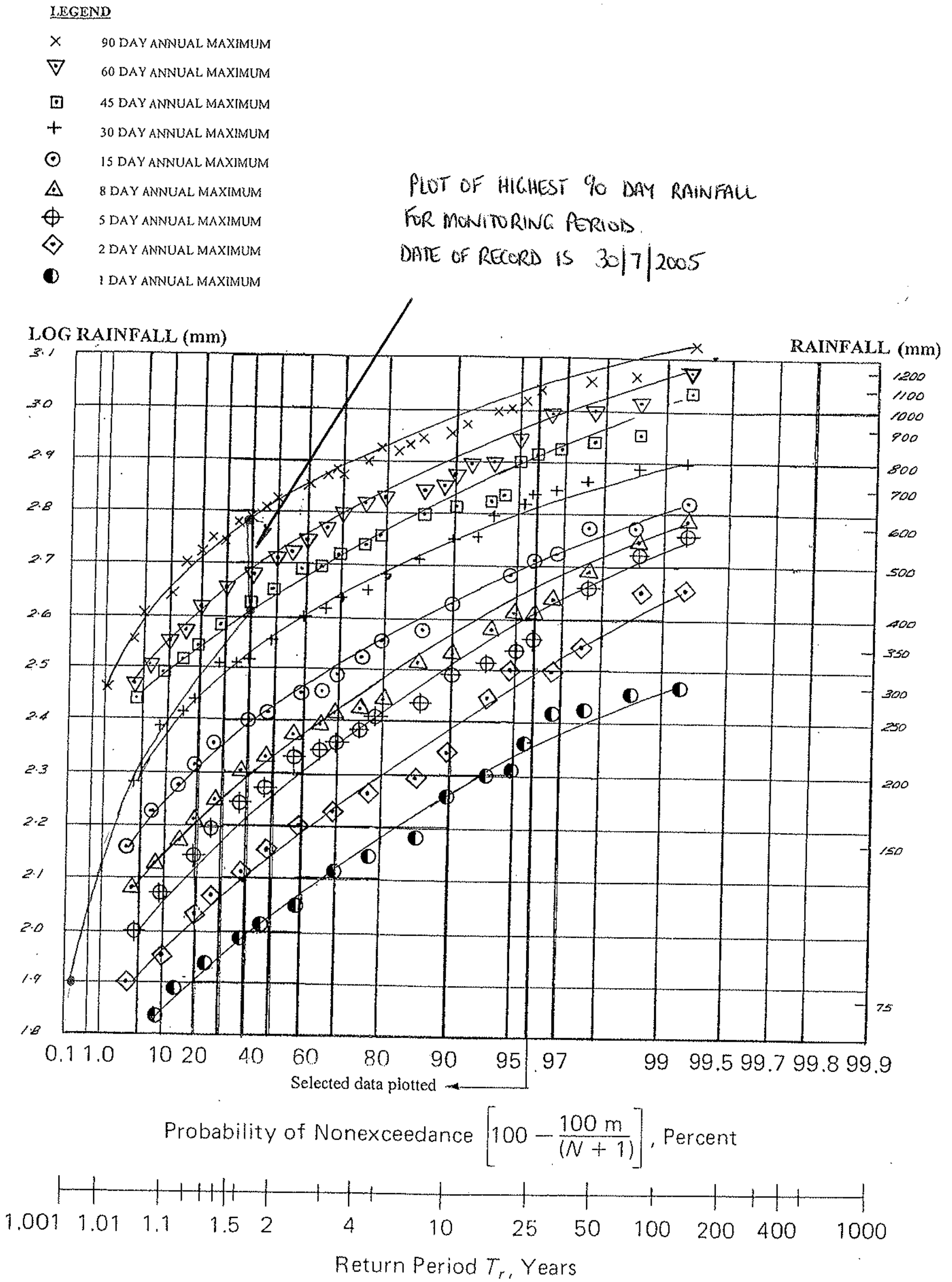
Report No. 19314WL Figure No. B2



PROBABILITY PLOT OF ACTUAL RAINFALL
AND ANTECEDANT RAINFALL FOR YAMBA

Jeffery & Katauskas Pty Ltd

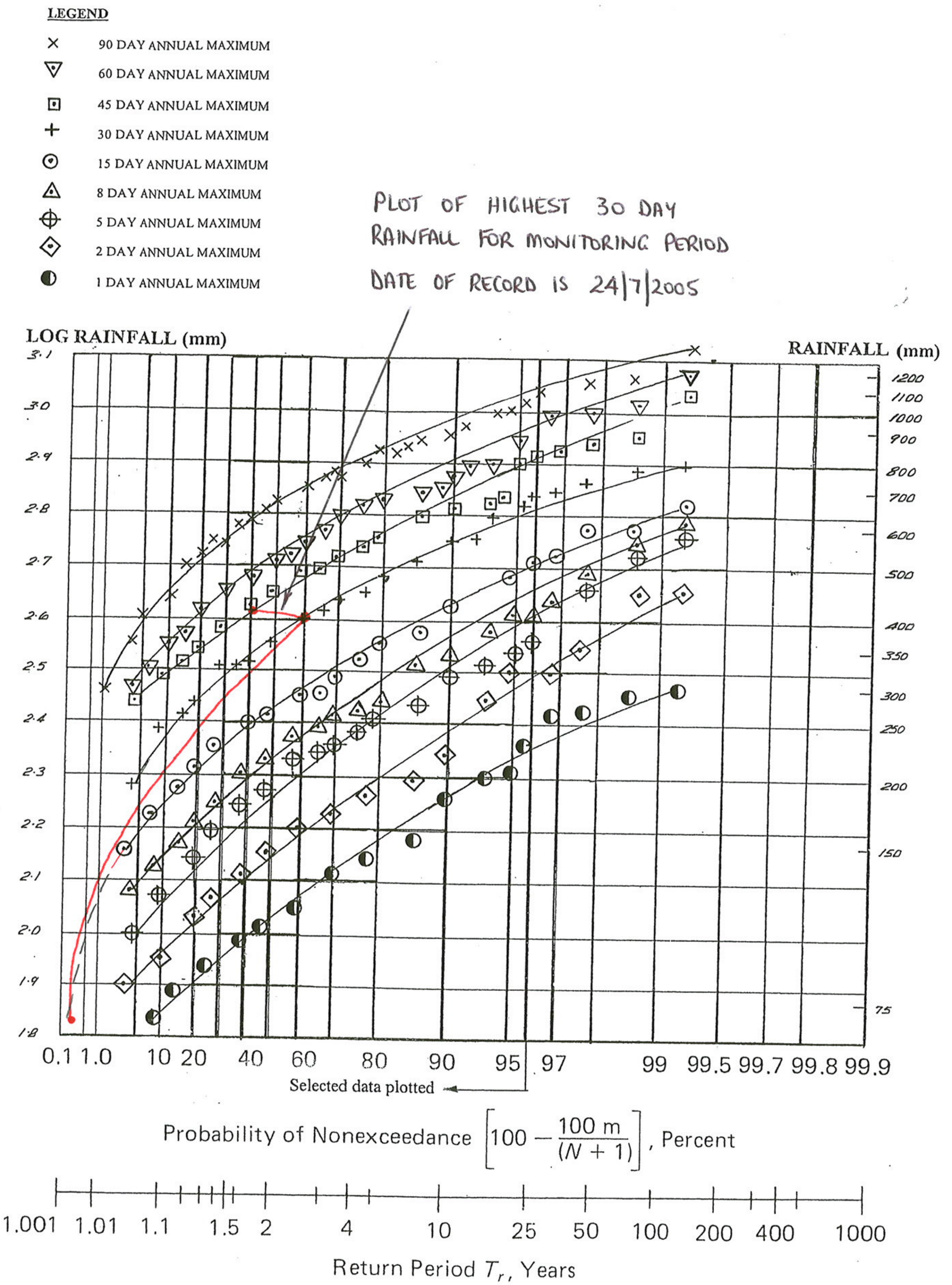
Report No. 19314WL Figure No. B2



PROBABILITY PLOT OF ACTUAL RAINFALL
AND ANTECEDANT RAINFALL FOR YAMBA

Jeffery & Katauskas Pty Ltd

Report No. 19314WL Figure No. B4



PROBABILITY PLOT OF ACTUAL RAINFALL AND ANTECEDANT RAINFALL FOR YAMBA

Jeffery & Katauskas Pty Ltd

Report No. 19314WL Figure No. B5

APPENDIX C

University of New South Wales
School of Civil & Environmental Engineering
Water Research Laboratory

Project: 04154 Yamba

Date: 4/26/05

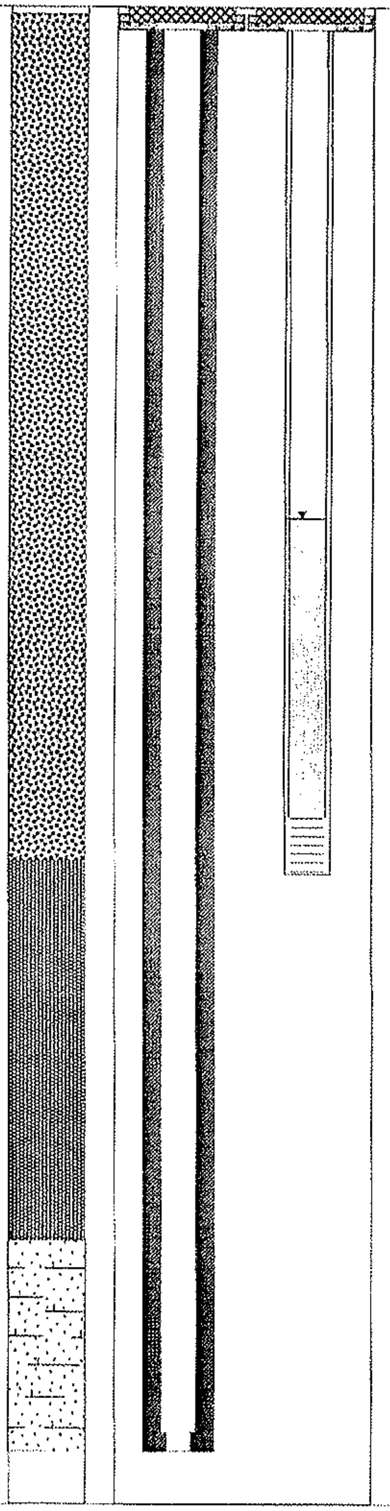
Hole No. 1A

Logged By: S.Pells

Driller: Shannon Simon of Craig Pulman Drilling P/L

Drill Hole Diameter: 100 mm

Method of Drilling: Inclinometer - Washbore to 18m, NMLC Core to 19.3m. Piezometer - washbore



Depth Below Ground (m)	Lithology	Construction comments
0	Topsoil	Completed ground level gattic covers
1		
2		
3	SAND. White-grey. Fine to medium grained. Subangular. Well sorted.	
4		
5		10.75m of Class 18 50 mm ID 64 mm OD uPVC Casing
6		
7		Water level at 6.83 m.b.g. Dip reading 29/04/05 13:38
8	SILTY SAND. Dark grey Iron cemented. Fine to medium grained. Some clay pieces.	
9		
10	SILTY SAND. Dark grey to black. Some clay.	
11		10.83 - 11.58 m: 0.75m of Class 18 50 mm ID 64 mm OD uPVC Machine Slotted Screen with filter sock
12	SANDY CLAY. Light Grey. Silty.	
13		
14		
15	SANDY CLAY. Grey / Orange / White. Silty.	
16		
17	SANDSTONE. Grey / Orange / White. Highly weathered residual clay.	
18	SANDSTONE	
19		19m of 70 mm OD uPVC Inclinometer Casing, grouted to surface annulus
20		

University of New South Wales
School of Civil & Environmental Engineering
Water Research Laboratory

Project: 04154 Yamba

Hole No. 1B

Driller: Hand Auger by Steven Pells

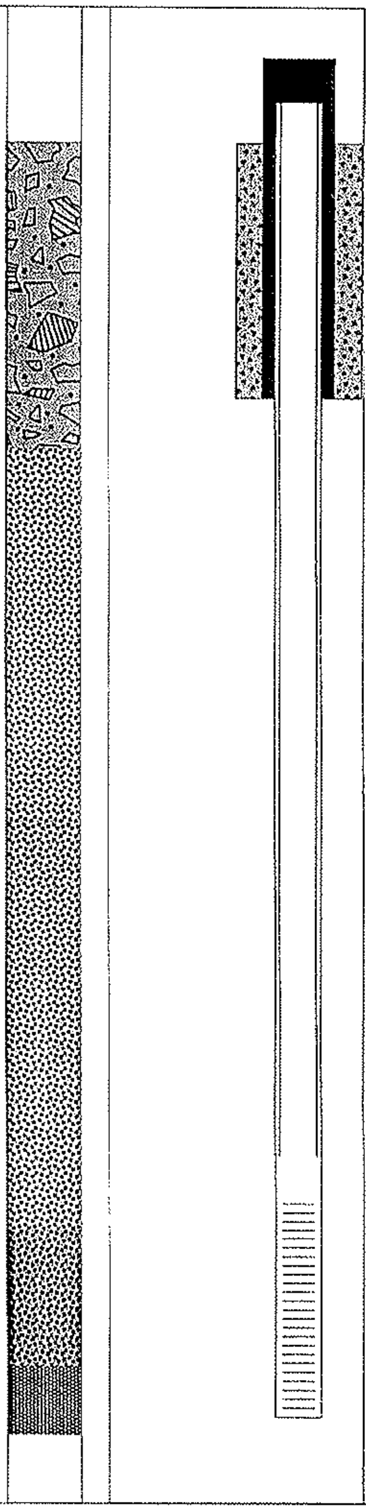
Method of Drilling: Hand Auger to 3.8m

Date: 4/28/05

Logged By: S.Pells

Drill Hole Diameter: 75 mm

Depth Below Ground (m)	Lithology	Graphic Log	Piezometer	Construction comments
0.0				Lockable steel monument
0.5	FILL. Silty Sand. Brown / Dark Grey. Fine-Medium grained. Some rubble			
1.0				
1.5	SAND. Light brown. Medium to Fine Grained. Occasional bands of bright orange (50mm). Some silt. Dry			3.1m of Class 18 50 mm ID 64 mm OD uPVC Casing
2.0	SILTY SAND. Dark Grey. Brown, iron cemented, fine - medium. Hard. Dry.			
2.5	SILTY SAND. Dark Grey. Iron Cemented or Indurated. Difficult to Auger. Dry			
3.0				
3.5	CLAYEY SAND. Orange / White clayey sand. Moist.			2.98 - 3.73 m: 0.75m of Class 18 50 mm ID 64 mm OD uPVC Machine Slotted Screen with filter sock.
	SANDY CLAY. White / Orange Mottled. Medium Plasticity. Sand Medium Grained to Coarse.			
4.0				



University of New South Wales
School of Civil & Environmental Engineering
Water Research Laboratory

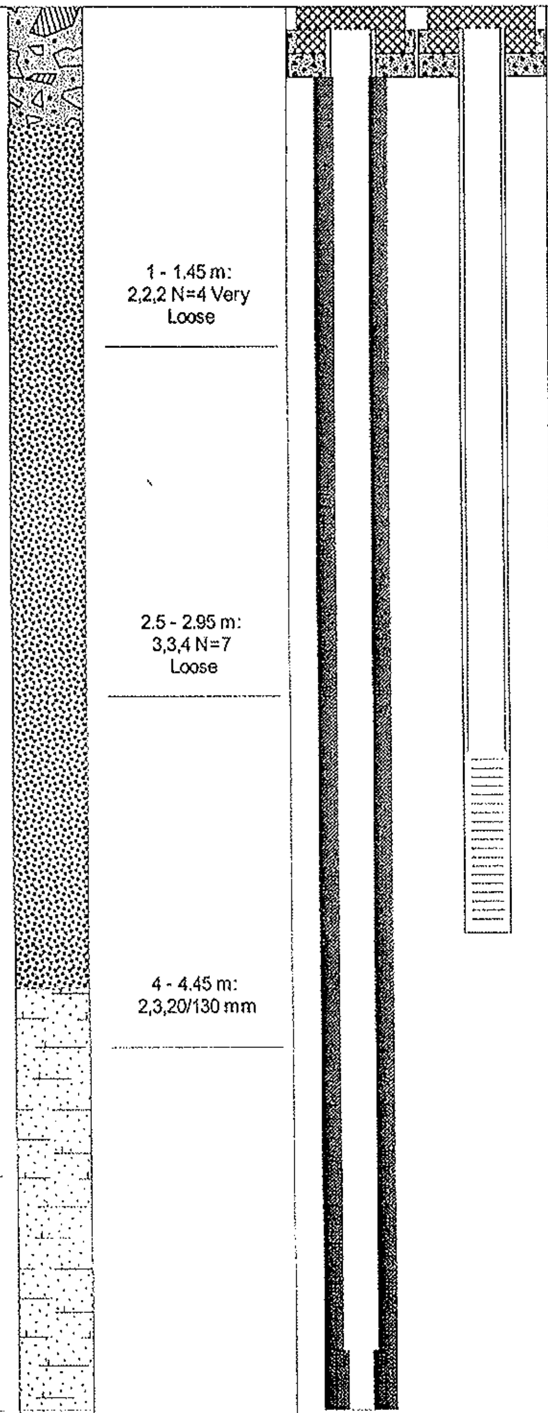
Project: 04154 Yamba | Date: 4/28/05

Hole No. 1C | Logged By: S.Pells

Driller: Shannon Simon of Craig Pulman Drilling P/L | Drill Hole Diameter: 100 mm

Method of Drilling: Auger

Depth Below Ground (m)	Lithology	Graphic Log	SPT Test	Inclinometer	Piezometer	Construction comments
0.0 – 0.5	FILL. Silty Sand. Brown / Dark Grey. Fine-Medium grained.					Completed ground level gattic covers
0.5 – 4.0	SAND. White / Grey. Medium to Fine Grained Beach.		1 - 1.45 m: 2,2,2 N=4 Very Loose 2.5 - 2.95 m: 3,3,4 N=7 Loose			3.1m of Class 18 50 mm ID 64 mm OD uPVC Casing 3.20 - 3.95 m: 0.75m of Class 18 50 mm ID 64 mm OD uPVC Machine Slotted Screen with filter sock.
4.0 – 4.2	SILTY SAND. Dark grey/brown iron cemented. Fine to medium grained. Moist.		4 - 4.45 m: 2,3,20/130 mm			
4.2 – 5.0	SANDSTONE. Highly weathered - sandy clay. White/orange/grey mottled					
5.0 – 6.0	SANDSTONE					~5.75m of 70 mm OD uPVC Inclinometer Casing, grouted to surface annulus



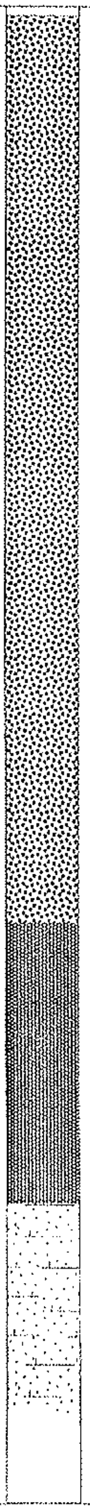
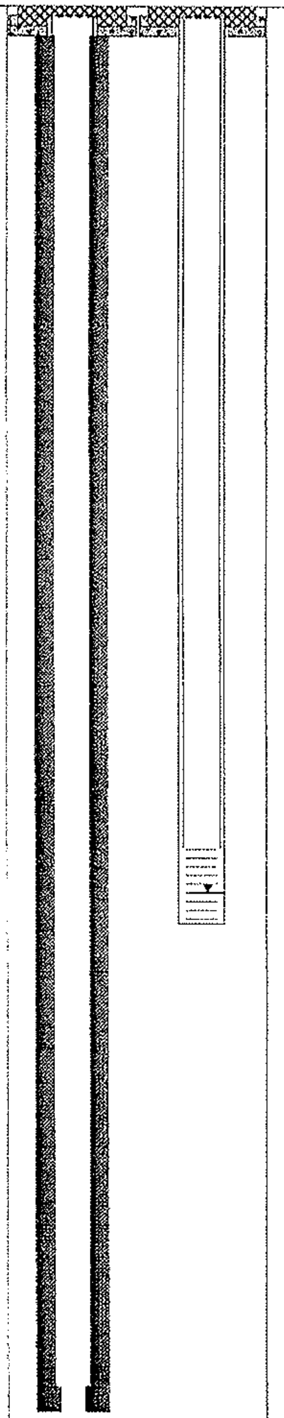
University of New South Wales
School of Civil & Environmental Engineering
Water Research Laboratory

Project: 04154 Yamba | Date: 27-Apr-05

Hole No. 2A | Logged By: S.Pells

Driller: Shannon Simon of Craig Pulman Drilling P/L | Drill Hole Diameter: 100 mm

Method of Drilling: Inclinometer - Washbore to 12m, NMLC Core to 14.1m. Piezometer - Washbore

Depth Below Ground (m)	Lithology	Graphic Log	SPT Test	Inclinometer	Piezometer	Construction comments
0	TOPSOIL					Completed ground level gattic covers
0 – 5.3	SAND. White / Grey. Medium to Fine Grained beach sand. Dry.		3 - 3.45 m: 4,5,8 N=13 Medium Dense			8.33m of Class 18 50 mm ID 64 mm OD uPVC Casing.
5.3 – 9.2	SILTY SAND.Dark grey iron cemented. Fine to medium grained.		6 - 6.45 m: 5,6,8 N=14 Medium Dense; 9 - 9.45 m: 5,7,8 N=15 Medium Dense			8.45 - 9.20 m: 0.75m of Class 18 50 mm ID 64 mm OD uPVC Machine Slotted Screen with filter sock. Water level at 8.9 m.b.g. Dip reading 29/04/05 13:30
9.2 – 12	SANDY CLAY. White / Orange Mottled Grey Weathered Sandstone. Very Stiff. Medium - High Plasticity. Sand Medium Grained to Coarse.					
12 – 14.1	SANDSTONE. Intact. Core from 12.5 to 14.1					13.74m of 70 mm OD uPVC Inclinometer Casing, grouted to surface annulus
15						

University of New South Wales
School of Civil & Environmental Engineering
Water Research Laboratory

Project:	04154 Yamba	Date:	4/28/05
Hole No.	2C	Logged By:	S.Pells
Driller:	Shannon Simon of Craig Pulman Drilling P/L	Drill Hole Diameter:	100 mm

Method of Drilling: Inclinometer - Auger to 4.7m, NMLC Core to 5.9m. Piezometer - Auger

Depth Below Ground (m)	Lithology	Graphic Log	SPT Test	Inclinometer	Piezometer	Construction comments
0.0	SILTY SAND. Grey. Fine to medium Grained. With Some Red-Brown High Plasticity Clay Pieces.					Completed ground level gattic covers
0.5	SAND. Yellow / Brown. Fine to Medium Grained.					0.55m of Class 18 50 mm ID 64 mm OD uPVC Casing
						0.64 - 1.39 m: 0.75m of Class 18 50 mm ID 64 mm OD uPVC Machine Slotted Screen with filter sock.
1.0	SANDY CLAY. White / Orange Mottled Grey. Firm to Very Stiff. Medium - High Plasticity. Becoming moist and less sandy with depth.		1 - 1.45 m: 2,2,2 N=4			
1.5						
2.0						
2.5			2.5 - 2.95 m: 3,5,4 N=9			
3.0						
3.5						
	SANDSTONE. Weathered.		4 - 4.05 m: Refusal			
4.0	SANDSTONE. Intact. Cored from 4.8 to 5.9 m					
4.5						
5.0						
5.5						5.7m of 70 mm OD uPVC Inclinometer Casing, grouted to surface annulus No End Valve Used
6.0						

University of New South Wales
School of Civil & Environmental Engineering
Water Research Laboratory

Project: 04154 Yamba | Date: 29/04/2005, 27/04/05

Hole No. 3A | Logged By: S.Pells

Driller: Shannon Simon of Craig Pulman Drilling P/L | Drill Hole Diameter: 100 mm

Method of Drilling: Inclinometer - Washbore to 10m, NMLC Core to 11.55m. Piezometer - Washbore

Depth Below Ground (m)	Lithology	Graphic Log	SPT Test	Inclinometer	Piezometer	Construction comments
0.0	TOPSOIL					Completed ground level gattic covers
0.0 – 1.75	SAND. Light Grey. Fine to medium Grained. With Some Silt		1 - 1.45 m: 2,2,3 N=5 Loose			
1.75 – 3.0	SAND. Light Grey. Fine to medium grained. Clay Fraction.		2.5 - 2.95 m: 5,6,10 N=16 Medium Dense			4.42m of Class 18 50 mm ID 64 mm OD uPVC Casing
3.0 – 5.2	SAND. Light Grey. Fine to medium Grained. Moisture at 5.0 m		4 - 4.45 m: 7,9,13 N=22 Medium Dense			Water level at 4.26 m.b.g. Dip reading 29/04/05 14:20 4.56 - 5.31 m: 0.75m of Class 18 50 mm ID 64 mm OD uPVC Machine Slotted Screen with filter sock.
5.2 – 8.5	SANDY CLAY. White / Orange Mottled Grey Weathered Sandstone. Very Stiff. Medium - High Plasticity. Sand Medium Grained to Coarse.		5.3 - 5.75 m: 2,5,7 N=12 7 - 7.45 m: 4,10,11 N=21 Medium Dense			
8.5 – 10.0	SANDSTONE. Highly weathered.		8.5 - 8.95 m: 4,15,17 N=32			
10.0 – 12.0						11.5m of 70 mm OD uPVC Inclinometer Casing, grouted to surface annulus

University of New South Wales
School of Civil & Environmental Engineering
Water Research Laboratory

Project: 04154 Yamba

Hole No. 3B

Driller: Hand Auger by Steven Pells

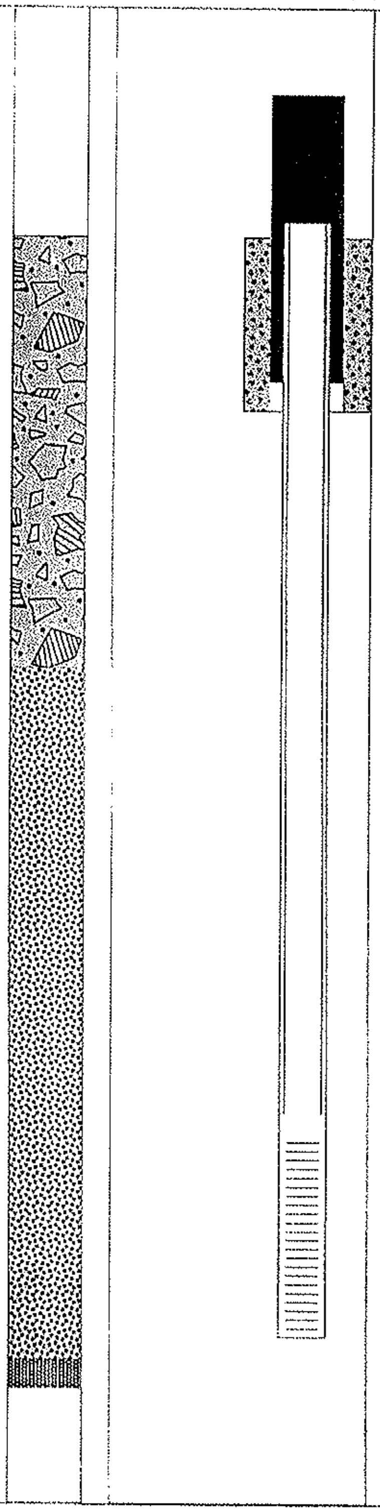
Method of Drilling: Hand Auger to 4.0m

Date: 28-Apr-05

Logged By: S.Pells

Drill Hole Diameter: 75 mm

Depth Below Ground (m)	Lithology	Graphic Log	Piezometer	Construction comments
0.5				
0.0				Lockable steel monument
0.5	FILL. Sand. Grey/white medium to fine. Dry. Pieces of Building Rubble.			
1.0				
1.5				3.1m of Class 18 50 mm ID 64 mm OD uPVC Casing
2.0				
2.5	SAND. White / Grey. Medium to Fine Grained. Dry			
3.0				
3.5				3.05 - 3.80 m: 0.75m of Class 18 50 mm ID 64 mm OD uPVC Machine Slotted Screen with filter sock.
	SILTY SAND. Dark Brown.			
4.0	SANDY CLAY. White/Orange Mottled. Stiff. Moist.			



University of New South Wales
School of Civil & Environmental Engineering
Water Research Laboratory

Project: 04154 Yamba

Date: 4/28/05

Hole No. 3C

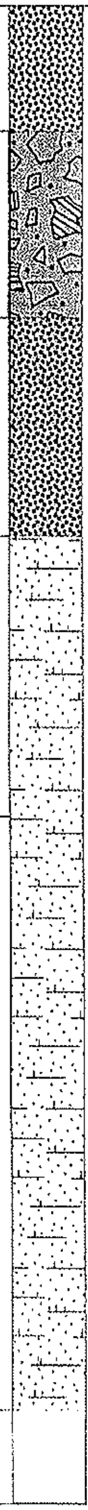
Logged By: S.Pells

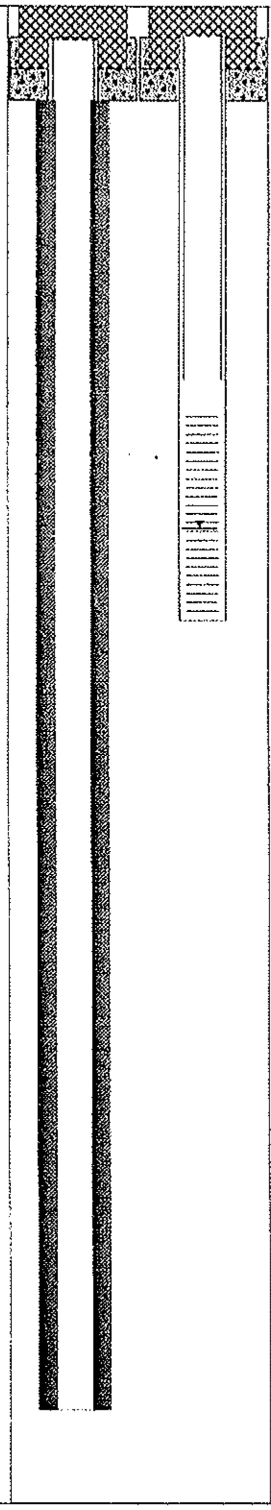
Driller: Shannon Simon of Craig Pulman Drilling P/L

Drill Hole Diameter: 100 mm

Method of Drilling: Auger

Depth Below Ground (m)	Lithology	Graphic Log	SPT Test	Inclinometer	Piezometer	Construction comments
0.0	SAND. Grey. Medium to fine.					Completed ground level gatlic covers
0.5	FILL. Brown. Silty Sand					1.1m of Class 18 50 mm ID 64 mm OD uPVC Casing
1.0	SILTY SAND. Dark to Black. Medium to Fine. Iron Cemented in bands. Fraction Silt.		1 - 1.45 m: 2,2,3 N=5 Loose			1.20 - 1.95 m: 0.75m of Class 18 50 mm ID 64 mm OD uPVC Machine Slotted Screen with filter sock.
1.5						Water level at 1.67 m.b.g. Dip reading 29/04/05 14:00
2.0	SANDSTONE. Interbedded Highly Weathered Sandstone / Clay. Yellow / Orange Mottled Grey		2.5 - 2.58 m: 24 / 80 mm - Refusal			4.14m of 70 mm OD uPVC Inclinometer Casing, grouted to surface annulus
2.5						
3.0						
3.5	SANDSTONE.					
4.0						
4.5						





APPENDIX D

TABLE C
SUMMARY OF KNOWN LANDSLIDES AND EVENTS

DATE	LOCATION	TYPE
1 & 2 March 1999	Pacific Hotel and Yamba Beach Craigmore Headland	Scour **Earthslides**
? / ? / 1996	YSLSC	Wave Attack
About 1 April 1994	Pacific Hotel	**Earthslides**
7 April 1988	Calypso Caravan Park (West side Yamba Hill)	***Earthflow***
Early 1977	Beer Garden of Pacific Hotel Craigmore Headland	**Earthslide** - creep movement **Earthslides**
March 1974	Pacific Hotel / north of YSLSC	**Earthslides**
6 February 1974	YSLSC	Beach scour & wave attack
7 or 8 April 1962	Hillside on drive to Yamba Beach	**Earthslide**
Late June & early July 1950	Pacific Hotel (destroyed) following after cyclone of 23 June 1950	**Earthslide**
? July 1950	West side Yamba hill	**Earthslide**
28 July 1950	Craigmore guesthouse	**Earthslide** / Scour
15 June 1945	YSLSC	Wave attack
About 25 or 26 May 1938	Yamba Beach hillside	Scour
14 & 15 May 1921	Yamba Beach hillside	Scour and/or **Earthslide**
About 6 August 1889	Flood damage to river walls	Not known

NOTE: Refer to Appendix A for further details.

TABLE D

ACTUAL RAINFALL AND ANTECEDANT RAINFALLS (mm) FOR KNOWN LANDSLIDES AND EVENTS

DATE	Daily	2 DAY	5 DAY	8 DAY	15 DAY	30 DAY	45 DAY	60 DAY	90 DAY
02 March 1999	300.0	379.6	444.4	473.0	500.2	656.0	721.8	786.8	870.2
06 June 1996	32.2	32.8	51.2	53.8	58.0	204.0	573.4	582.4	685.0
31 March 1994	101.0	106.9	177.1	328.2	348.0	548.8	617.4	665.8	705.6
01 April 1994	16.8	117.8	167.9	297.6	364.8	549.6	632.2	675.8	722.4
07 April 1988	137.0	159.0	248.2	365.4	444.2	630.0	762.7	916.7	1066.7
06 October 1982	0.0	0.0	0.0	0.0	57.9	208.1	240.7	302.5	395.9
22 February 1977	122.0	133.0	133.0	133.4	160.8	211.3	229.7	334.2	430.7
03 March 1977	82.4	85.3	122.8	144.3	287.0	365.3	365.3	447.5	540.3
19 May 1977	134.4	216.2	335.0	340.4	354.0	447.4	493.7	508.4	846.4
01 February 1974	35.4	38.6	46.2	95.4	99.9	223.3	312.7	396.1	455.1
06 February 1974	12.6	12.8	12.8	51.7	108.2	170.2	255.4	344.9	448.0
10 March 1974	173.0	173.0	175.6	177.6	188.4	241.4	359.6	419.0	590.9
11 March 1974	287.2	460.2	462.8	463.6	475.6	527.9	640.5	683.8	877.3
07 April 1962	199.6	228.0	253.7	293.9	304.8	466.9	523.5	564.5	995.7
08 April 1962	117.6	317.2	345.6	411.0	420.1	580.2	641.1	677.3	1000.8
23 June 1950	29.2	68.8	176.5	261.4	284.0	298.5	322.2	435.5	776.0
24 June 1950	95.5	124.7	203.2	352.8	379.5	391.2	411.3	531.0	865.4
25 June 1950	67.6	163.1	238.3	414.0	447.1	458.8	478.9	598.6	931.0
10 July 1950	103.6	104.4	104.4	104.4	107.4	554.5	566.2	586.3	965.7
28 July 1950	6.4	6.4	27.0	107.5	336.4	465.4	915.5	927.2	1049.2
29 July 1950	136.7	143.1	163.7	179.2	450.0	602.1	1046.6	1063.9	1177.3
15 June 1945	9.1	11.6	231.8	318.9	403.3	521.9	532.8	613.8	730.9
22 May 1938	91.9	119.3	137.1	149.8	236.4	311.9	421.9	489.2	565.8
23 May 1938	26.7	118.6	163.8	174.5	257.8	338.6	429.3	512.1	582.1
24 May 1938	42.2	68.9	206.0	214.9	299.2	380.8	458.8	547.2	624.3
25 May 1938	37.3	79.5	225.5	243.3	332.7	418.1	449.1	583.2	661.6
26 May 1938	14.2	51.5	212.3	257.5	346.9	432.3	455.2	594.9	675.8
14 July 1938	20.3	39.6	71.9	114.6	152.2	185.7	251.8	544.9	731.9
14 May 1921	133.6	135.9	151.1	190.8	233.0	334.4	475.5	510.1	620.9
15 May 1921	273.1	406.7	411.8	463.9	485.5	607.5	745.0	783.2	894.0
16 May 1921	187.5	460.6	599.3	622.4	660.3	793.5	926.9	968.4	1081.5
05 Aug 89	87.6	87.6	111.0	111.0	117.1	423.7	500.9	503.4	630.6
06 Aug 89	231.1	318.7	342.1	342.1	347.9	654.8	732.0	734.5	842.4
07 Aug 89	91.2	322.3	433.0	433.3	434.3	746.0	823.2	825.7	933.1

TABLE E
SUMMARY OF RETURN PERIODS OF ACTUAL RAINFALL AND ANTECEDENT RAINFALL FOR LANDSLIDE EVENTS

Date	Return Period (years) for Rainfall Over						Comments
	1 day to 2 day	5 day to 15 day	30 day to 45 day	60 day to 90 day	Critical Rainfall Period	Indicative Return Period (years)	
A. SCOUR EVENTS							
2 March 1999	124 to 60	40 to 25	25 to 15	9 to 6	1 day	124	Most rainfall reported over about 4 hours. Not consistent with other events.
26 May 1938	≈1	3 to 4	3 to 2	3 to 2	15 day	4	
15 & 16 May 1921	70 to 124	130 to 120	120 to 50	27 to 12	5 day	130	
B. EARTH SLIDE EVENTS							
31 March 1994	2 to 1	2 to 6	9 to 7	4 to 2	30 day	9	
19 May 1977	3 to 7	12 to 5	4 to 3	2 to 5	5 day	12	
11 March 1974	90 to 124	45 to 15	≈7	5 to 6	2 day	124	
7 April 1962	14 to 8	5 to 3	≈4	2 to 11	1 day	14	
8 April 1962	2 to 26	16 to 9	12 to 8	5 to 12	2 day	26	
25 June 1950	1 to 3	4 to 15	4 to 2	3 to 8	8 day	15	
10 July 1950	2 to 1	≈1	9 to 4	3 to 10	90 day (30 day)	10 9	
29 July 1950	4 to 2	1 to 10	15 to 100	50 to 35	45 day	100	
6 & 7 August 1889	25 to 30	30 to 10	65 to 25	11 to 9	30 day	65	
C. EARTHFLOWS							
7 April 1988	4 to 3	4 to 12	≈20	20 to 18	30 & 60 day	20	

11 September, 2007
Ref: 19314WLYamba TabF

TABLE F
RISK ESTIMATES FOR LOSS OF LIFE

No	Consideration/ Conditional Probability	LANDSLIDE RISK ZONE							
		1a		1b		2		1c & 3	
		Values	Comment	Values	Comment	Values	Comment	Values	Comment
I	Probability of Landsliding	$5x10^{-2}$ to $5x10^{-3}$	From rainfall and historical data	$5x10^{-2}$ to $5x10^{-3}$	From rainfall and historical data	10^{-2} to 10^{-3}	As not within area of reported slides but creep effects evident.	$5x10^{-2}$ to $4x10^{-3}$	From rainfall and historical data
II	Element at Risk	Pacific Hotel		Residential dwelling in area close to hotel		Residential dwellings in area of no known landslides		Undeveloped toe slopes above outcrop or foot of main hillside	
III	Probability of affecting Element at Risk	0.5 to 1.0 respectively	Assumes lower prob event likely to be larger, plus cumulative effects of upslope regression	0.2	Assumes 10m to 20m wide landslide over about 70m of slope, say 3 to 7 potential slides, on average 5, each about width of dwelling	0.1	Similar to B, but dwellings on flatter crest slopes	No. of potential slides = $^{500}/_{20}$ = 25. Probability of person at slide site = $^{1}/_{25}$ = $4x10^{-2}$	For person at landslide site, assumes about 20m wide landslides over 500m length of slope; non over-lapping, all equally likely.
IV	Likely rate of Movement and Probability	(a) Very Slow to Moderate 1.0 (b) Rapid to Very Rapid 0.2	Physical and Historical evidence Possible, but may be only near surface	(a) Very Slow to Moderate 1.0 (b) Rapid to Very Rapid 0.1	Physical and Historical evidence. Possible, but may be only near surface.	(a) Very Slow to Moderate 1.0 (b) Rapid to Very Rapid 0.05	Historical evidence. Less likely than 1b since further from instability.	(a) Very Slow to Moderate 1.0 (b) Rapid to Very Rapid, 1.0	Area likely to be affected by both scour and earthslides
V	Probability of significant structural damage	(a) 0.2 to 0.5, say 0.3 (b) 0.5 to 1.0, say 0.8		(a) 0.2 to 0.5, say 0.3 (b) 0.5 to 1.0, say 0.8		(a) 0.1 to 0.4, say 0.2 (b) 0.4 to 0.8, say 0.7	Reduced from 1b due to flatter crest slopes. May not affect much of dwelling.	N/A	
VI	Affect on Element	for (a) for (b)	Cracking and distortion, with time becomes unsafe/unusable. Rapid cracking, possible collapse.	for (a) for (b)	As 1a Rapid cracking, possible collapse. For dwellings at lower elevation, possible impact from above.	for (a) for (b)	As for 1a As for 1a	N/A	
VII	Vulnerability to Persons in area affected	for (a) 0.01 for (b) 0.8 to 1.0, say 1.0	Escape due to warning by cracking likely, some may be "unlucky" Escape may not be possible.	for (a) 0.01 for (b) 0.8 to 1.0, say 1.0	Escape due to warning by cracking likely, some may be "unlucky". Escape may not be possible, may be buried.	for (a) 0.01 for (b) 1.0	As for 1a As for 1a	for (a) 0.01 for (b) 0.5	As for 1a Assumes 50% chance of not being buried.

TABLE F (continued)

No	Consideration/ Conditional Probability	LANDSLIDE RISK ZONE							
		1a		1b		2		1c & 3	
		Values	Comment	Values	Comment	Values	Comment	Values	Comment

VIII	Occupancy/ Temporal Probability for person most at risk	0.7 to 1.0,
		say 1.0
			For person staying in Hotel accommodation and using bar & restaurant.
			Failure more likely during inclement weather therefore prolonged occupancy.
				for (a) 0.7
				
				for (b) 0.9
					Assumes persons absent on average ≈8 hours/day.
					
					Assumes persons more likely to be present during inclement weather.
						for (a) 0.25
						
						for (b) 0.5
							Assumes area affected not bedrooms, living area occupied about 6 hours/day.
							Assumes person more likely to be present during inclement weather, but bedrooms not affected.
								$^{0.5}/_{24}$ to $^{1}/_{24}$
								= 0.02 to 0.04
									Assumes person is regular user, walking through area every day.
									Occupancy assumed for 2 hour to 1 hour per day.

IX	Risk Estimate for person most at risk	for (a) 5×10^{-2} x 0.5x 0.3 x 1.0 x 0.01 x 1.0 **= 7.5 x 10^{-5}** to 5×10^{-3} x 1.0 x 0.3 x 1.0 x 0.01 x 1.0 **= 1.5 x 10^{-5}**	Very Slow to Moderate movements	for (a) 5×10^{-2} x 0.2 x 1.0 x 0.3 x 0.01 x 0.7 **= 2 x 10^{-5}** to 5×10^{-3} x 0.2 x 1.0 x 0.3 x 0.01 x 0.7 **= 2 x 10^{-6}**	Very Slow to Moderate movements	for (a) 10^{-2} x 1.0 x 0.2 x 0.01 x 0.25 **= 5 x 10^{-6}** to 10^{-3} x 1.0 x 0.2 x 0.01 x 0.25 **= 5 x 10^{-7}**	Very Slow to Moderate movements	For (a) 5×10^{-2} x 4×10^{-2}x 1.0 x 0.01 x (0.02 to 0.04) **= 4×10^{-7} to 8×10^{-7}** to 4×10^{-3} x 4×10^{-2} x 1.0 x 0.01 x (0.02 to 0.04) **= 3.2×10^{-8} to 6.4×10^{-8}**	Very Slow to Moderate movements
		for (b) 5×10^{-2}x 0.5 x 0.8 x 0.1 x 1.0 x 1.0 **= 2 x 10^{-3}** to 5×10^{-3} x 1.0 x 0.8 x 0.1 x 1.0 x 1.0 **= 4 x 10^{-4}**	Rapid to Very Rapid movements	for (b) 5×10^{-2} x 0.2 x 0.1 x 0.8 x 1.0 x 0.9 **= 7.2 x 10^{-4}** to 5×10^{-3} x 0.2 x 0.1 x 0.8 x 1.0 x 0.9 **= 7.2 x 10^{-5}**	Rapid to Very Rapid movements	for (b) 10^{-2} x 0.1 x 0.05 x 0.7 x 1.0 x 0.5 **= 1.8 x 10^{-5}** to 10^{-3} x 0.1 x 0.05 x 0.7 x 1.0 x 0.5 **= 1.8 x 10^{-6}**	Rapid to Very Rapid movements	For (b) 5×10^{-2} x 4×10^{-2} x 1.0 x0.5 x (0.02 to 0.04) **= 2×10^{-5} to 4×10^{-5}** to 4×10^{-3} x 4×10^{-2} x 1.0 x 0.5 x (0.02 to 0.04) **= 1.5×10^{-6} to 3×10^{-6}**	Rapid to Very Rapid movements

APPENDIX B

INCLINOMETER MONITORING REPORTS BY JEFFERY AND KATAUSKAS PTY LTD

spare

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

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19 August 2005
Ref: 19314WLY Let

Water Research Laboratory
School of Civil and Environmental Engineering
University of New South Wales
King Street
MANLY VALE NSW 2093

ATTENTION: Dr Wendy Timms

Dear Wendy

INCLINOMETER MONITORING
PILOT HILL, YAMBA, NSW

This report presents the results of the second readings taken from the inclinometers installed at Pilot Hill, Yamba. The readings were completed on the 22nd July 2005 in each of the installed inclinometers. The location of each inclinometer is shown on the attached Figure 1.

Following the installation and grouting of the inclinometers between the 27th and 29th April 2005, baseline readings were taken no earlier than 12 hours following grouting. These baseline readings form the reference point from which all subsequent readings are measured. Consequently, all measured movement will be in relation to the state of the hillside at the time the baseline readings were recorded.

The results of these readings taken on the 22nd July 2005 indicate that at this stage no significant movement has occurred in relation to the baseline readings. Some movements were recorded however these were in the order of millimetres and are most likely due to 'settling in' of the casing.

The attached Figures 2 to 7 present these results. The next set of inclinometer readings is planned for the end of October 2005.

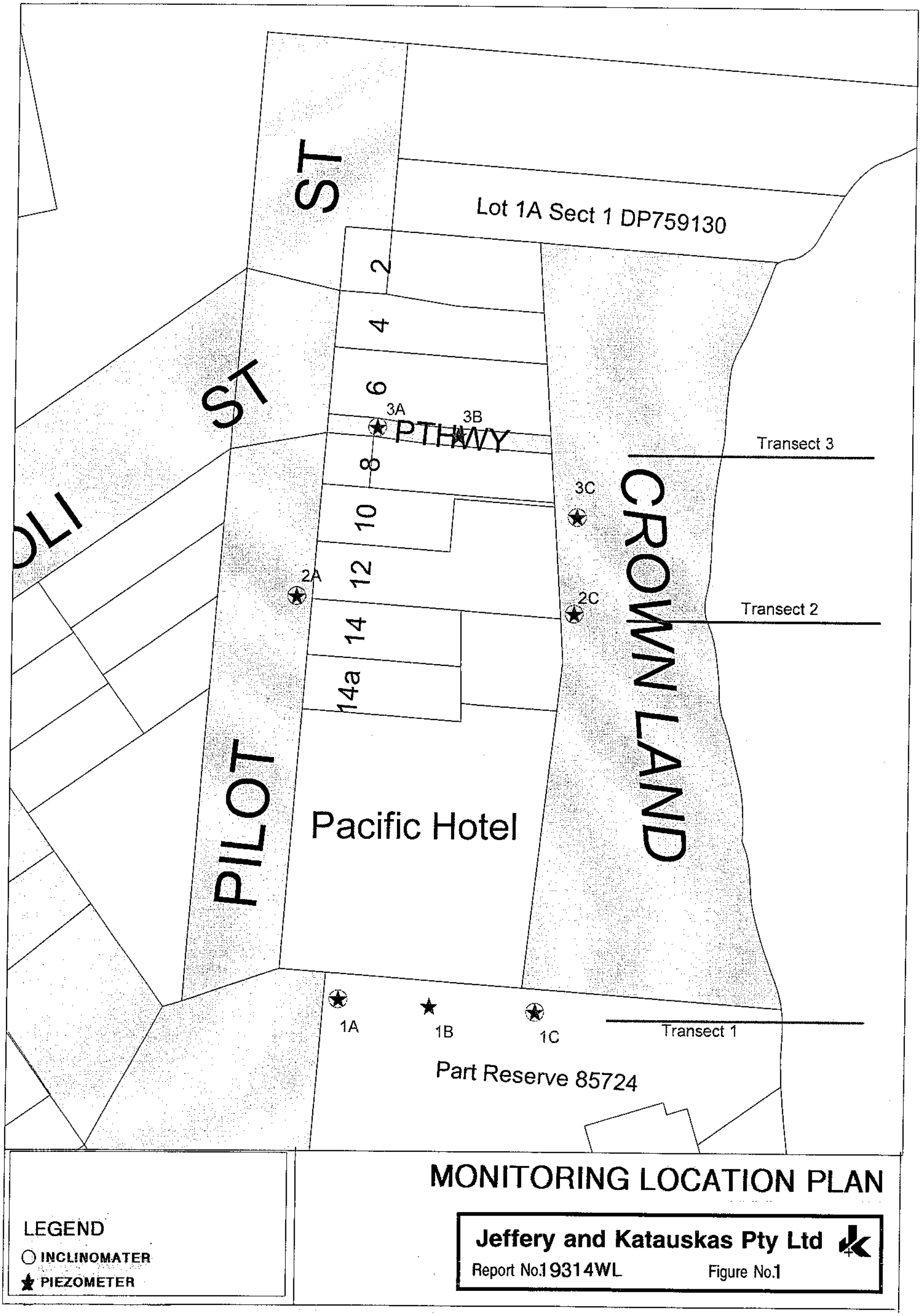
Should you require any further information regarding the above please do not hesitate to contact the undersigned.

Yours faithfully
For and on behalf of
JEFFERY AND KATAUSKAS PTY LTD

for W Theunissen
Senior Geotechnical Engineer

L Speechley
Senior Associate.

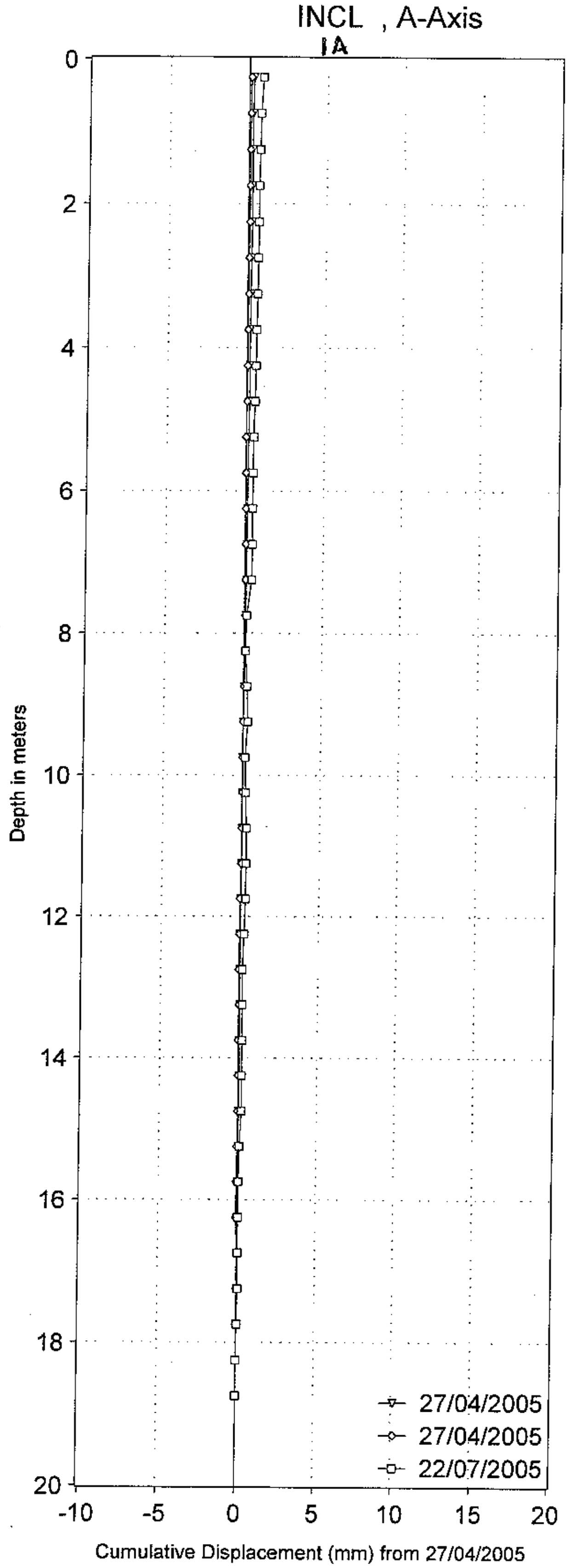
Enclosed: Figure 1: Inclinometer Location Plan
Figures 2 to 7: Inclinometer Test Results

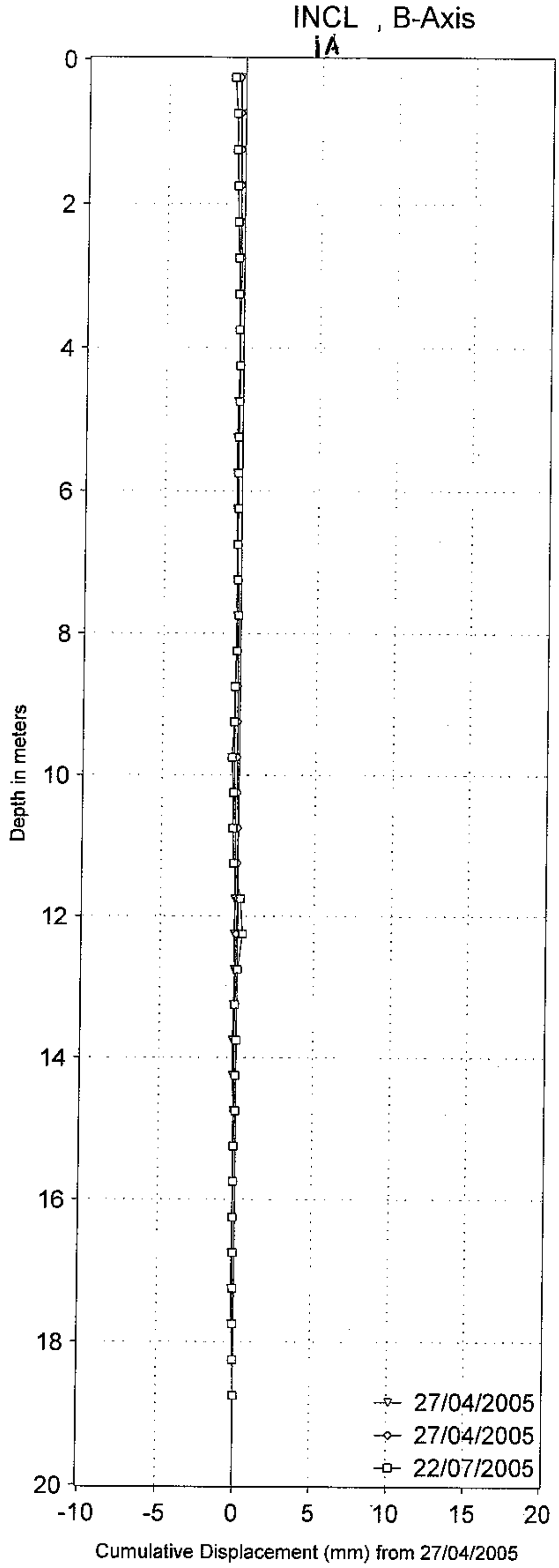


MONITORING LOCATION PLAN

Jeffery and Katauskas Pty Ltd

Report No.19314WL Figure No.1

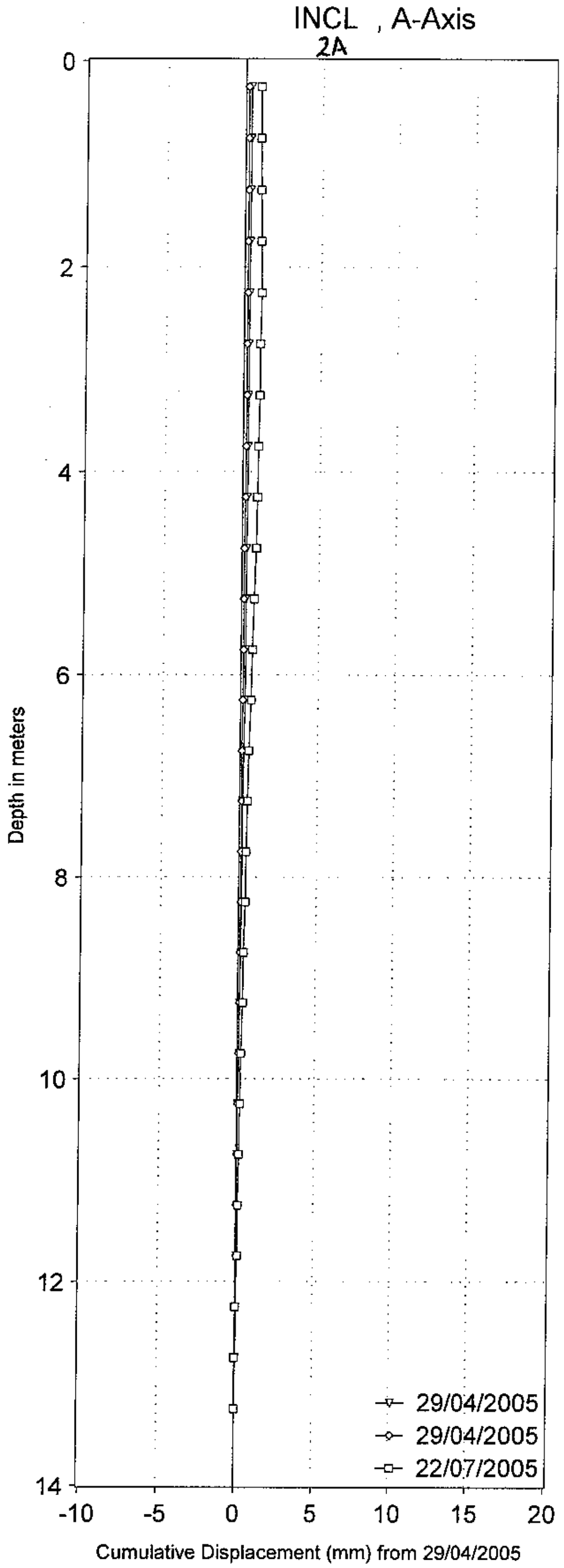


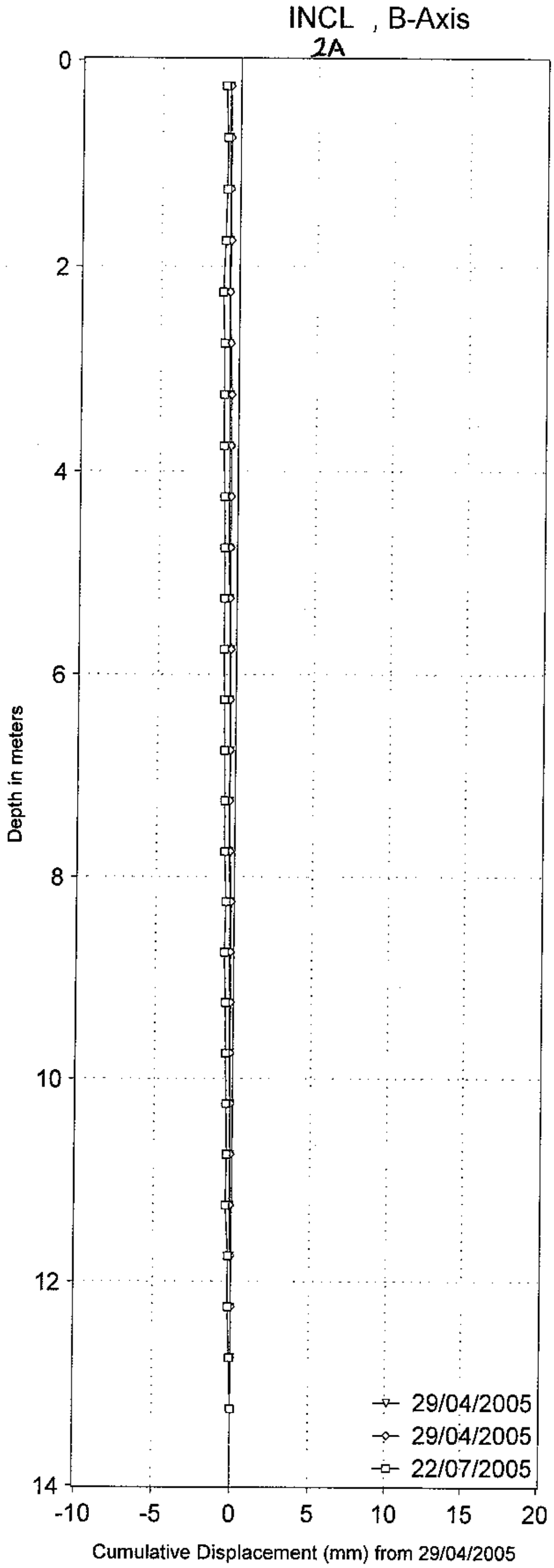


Jeffery and Katauskas Pty Ltd
CLIENT: Water Research Laboratory UNSW
PROJECT: Inclinometer Monitoring
LOCATION: Pilot Hill, Yamba, NSW

Job No: 19314WL
BH/INCL No: INCL1A

Figure No:2





Jeffery and Katauskas Pty Ltd

CLIENT: Water Research Laboratory UNSW

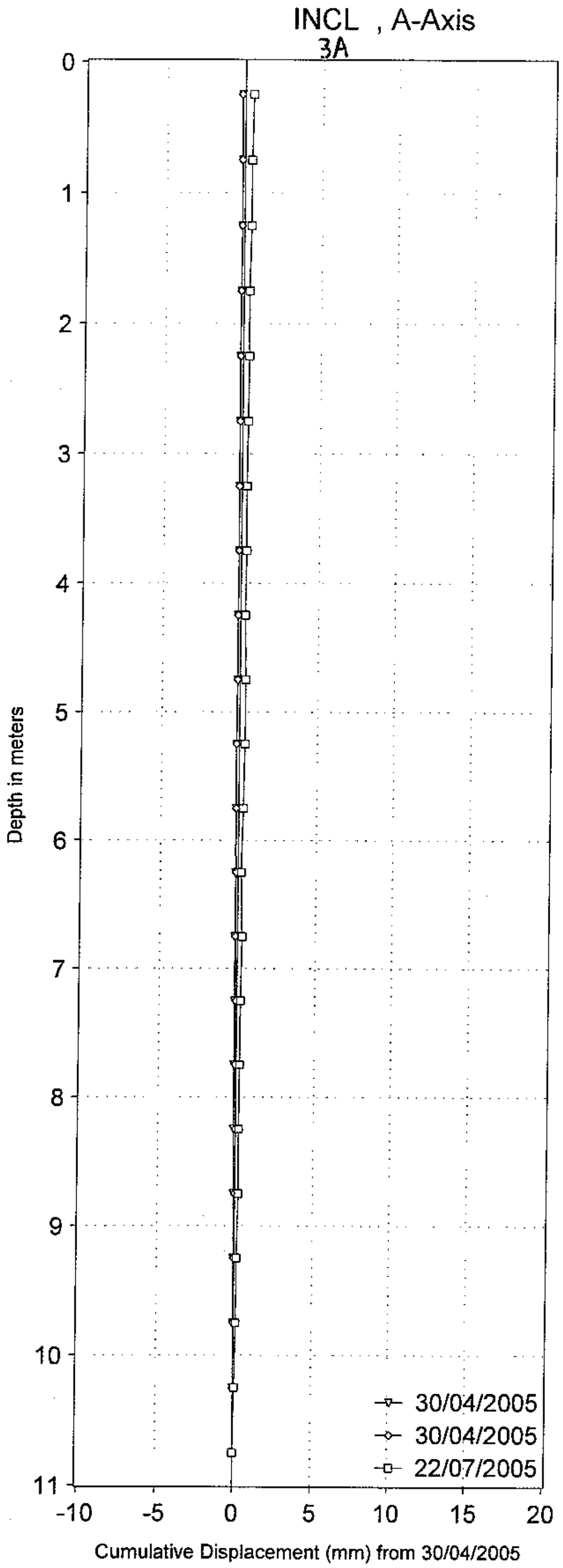
PROJECT: Inclinometer Monitoring

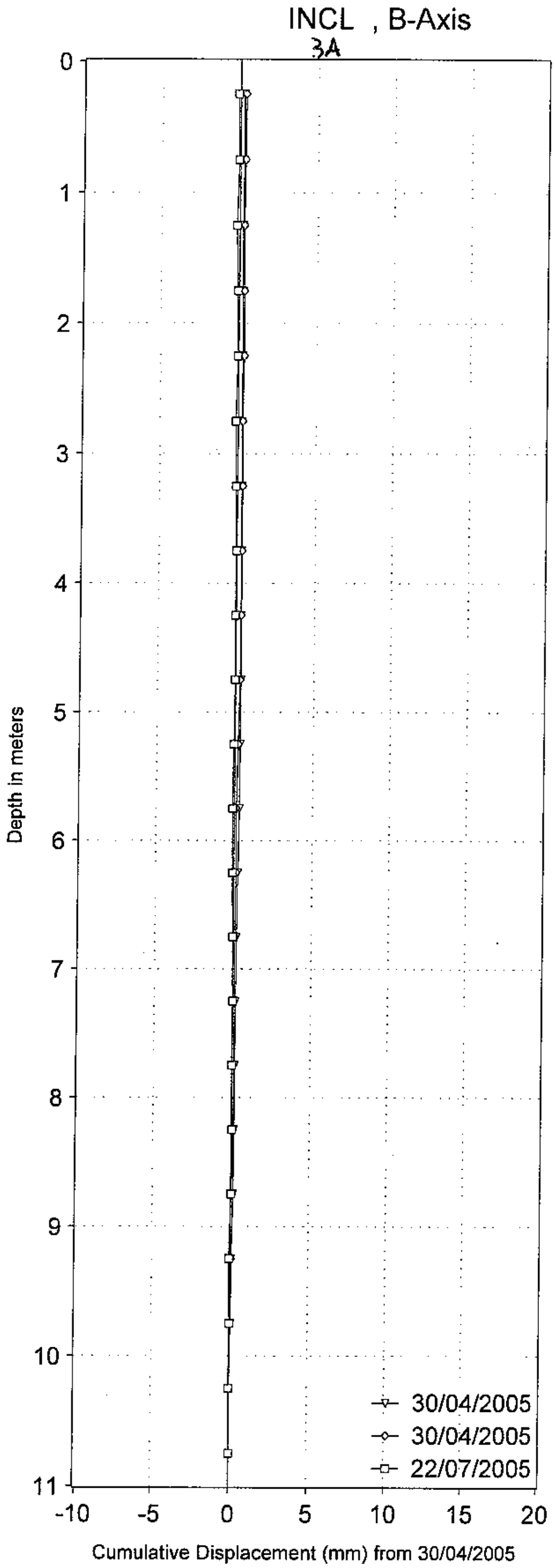
LOCATION: Pilot Hill, Yamba, NSW

Job No: 19314WL

BH/INCL No: INCL2A

Figure No: 3





Jeffery and Katauskas Pty Ltd

CLIENT: Water Research Laboratory UNSW

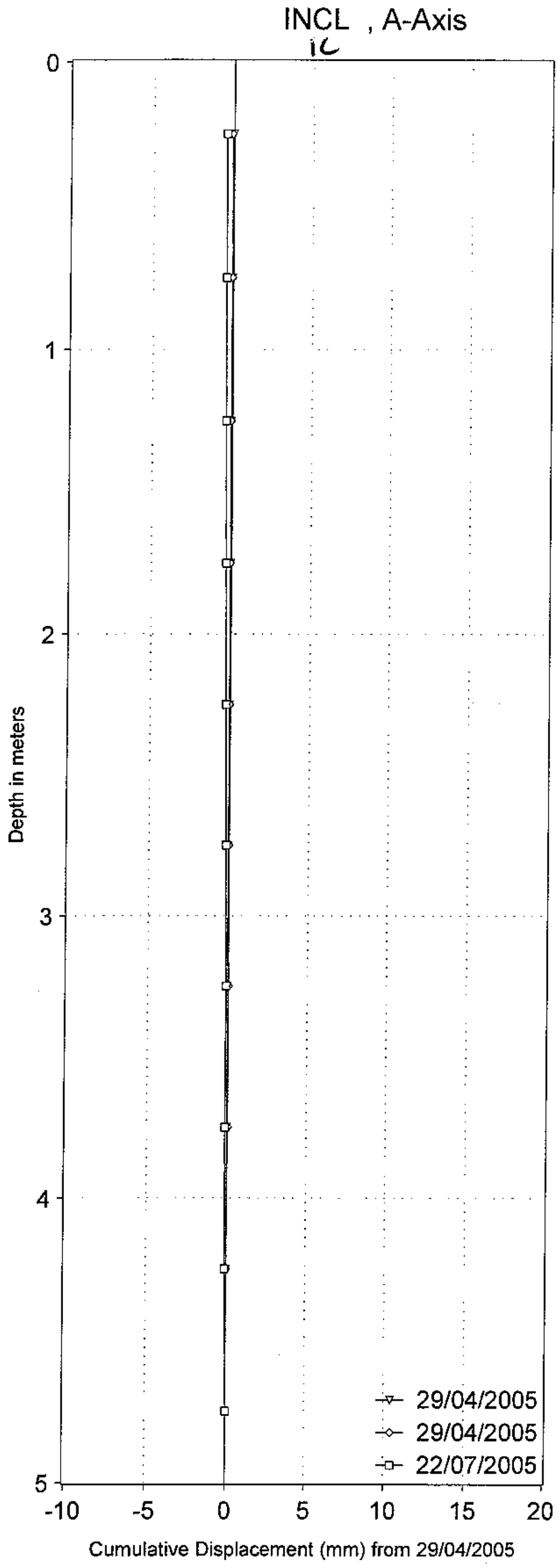
PROJECT: Inclinometer Monitoring

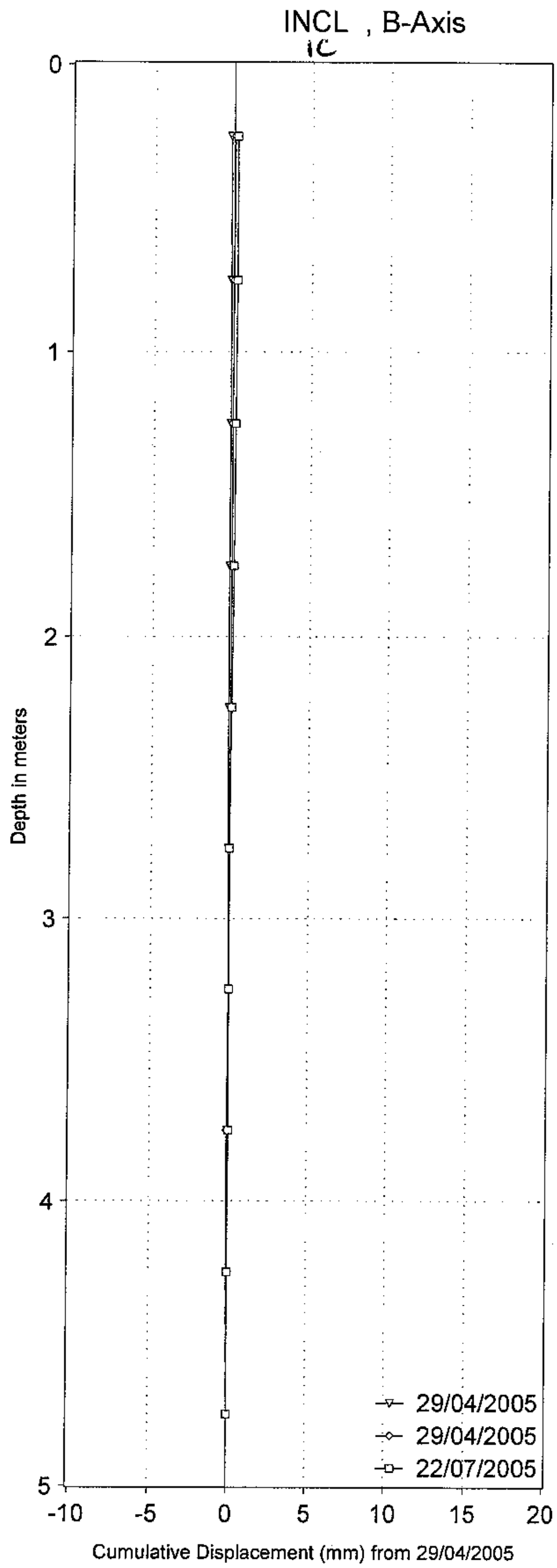
LOCATION: Pilot Hill, Yamba, NSW

Job No: 19314WL

BH/INCL No: INCL3A

Figure No: 4





Jeffery and Katauskas Pty Ltd

CLIENT: Water Research Laboratory UNSW

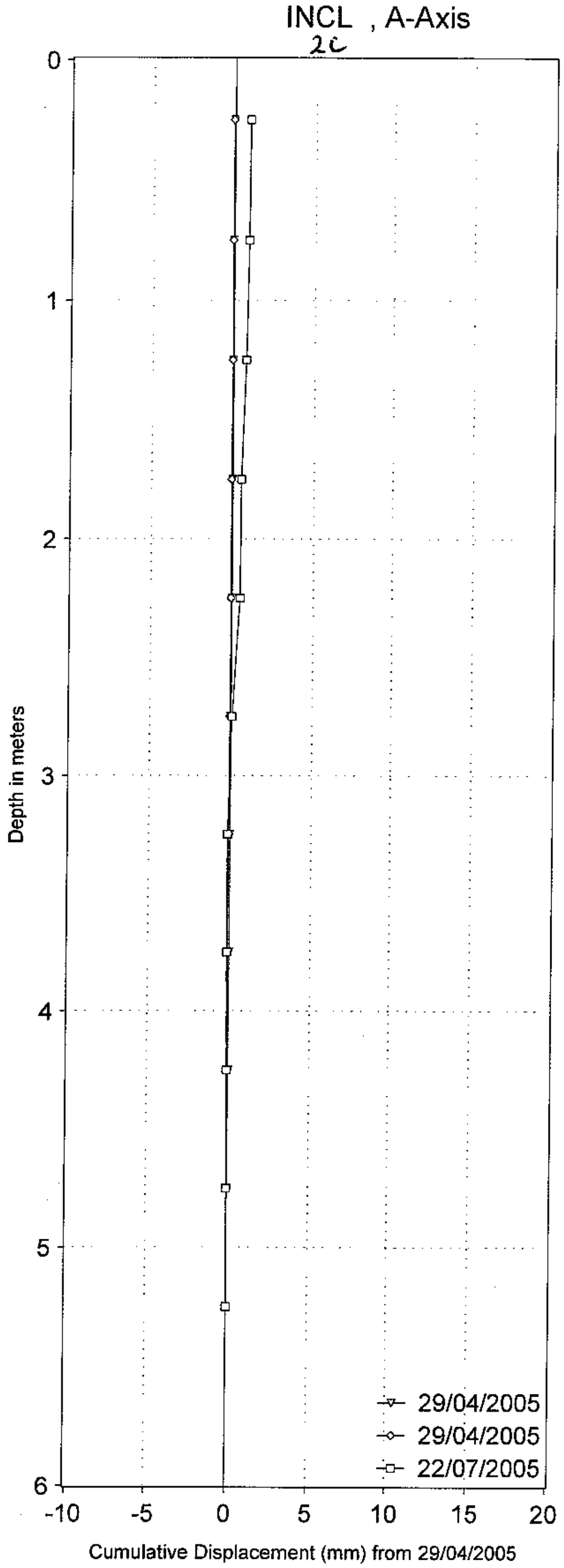
PROJECT: Inclinometer Monitoring

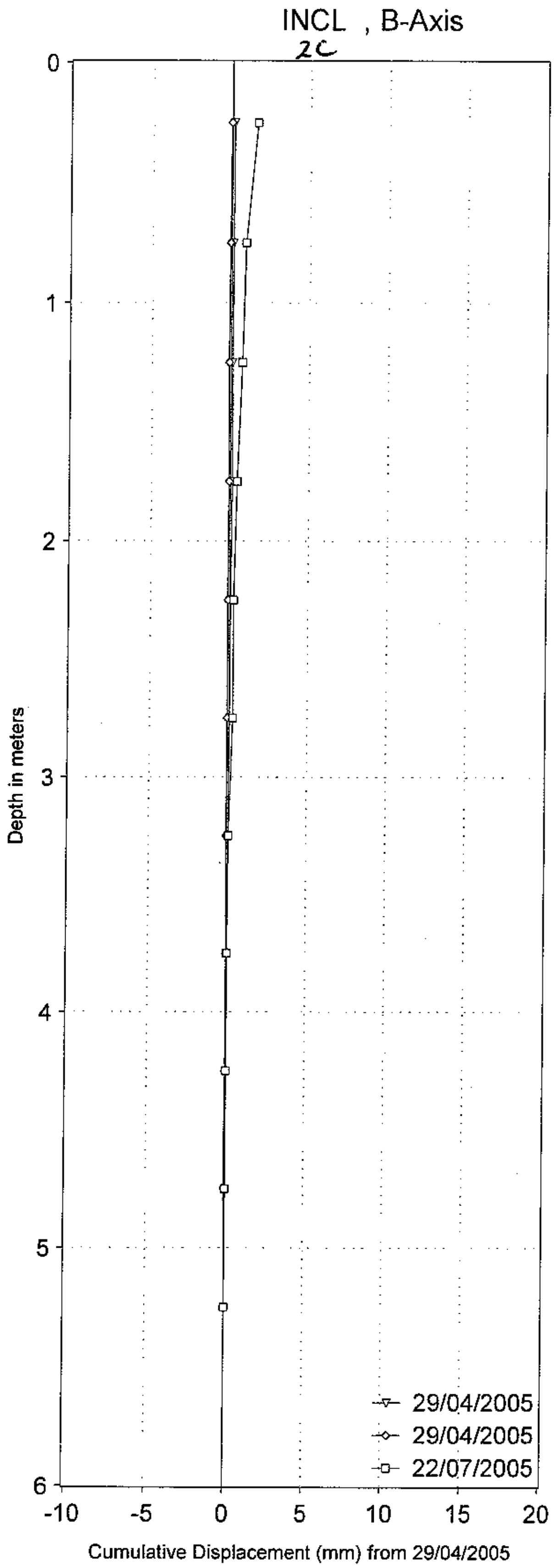
LOCATION: Pilot Hill, Yamba, NSW

Job No: 19314WL

BH/INCL No: INCL1C

Figure No:5

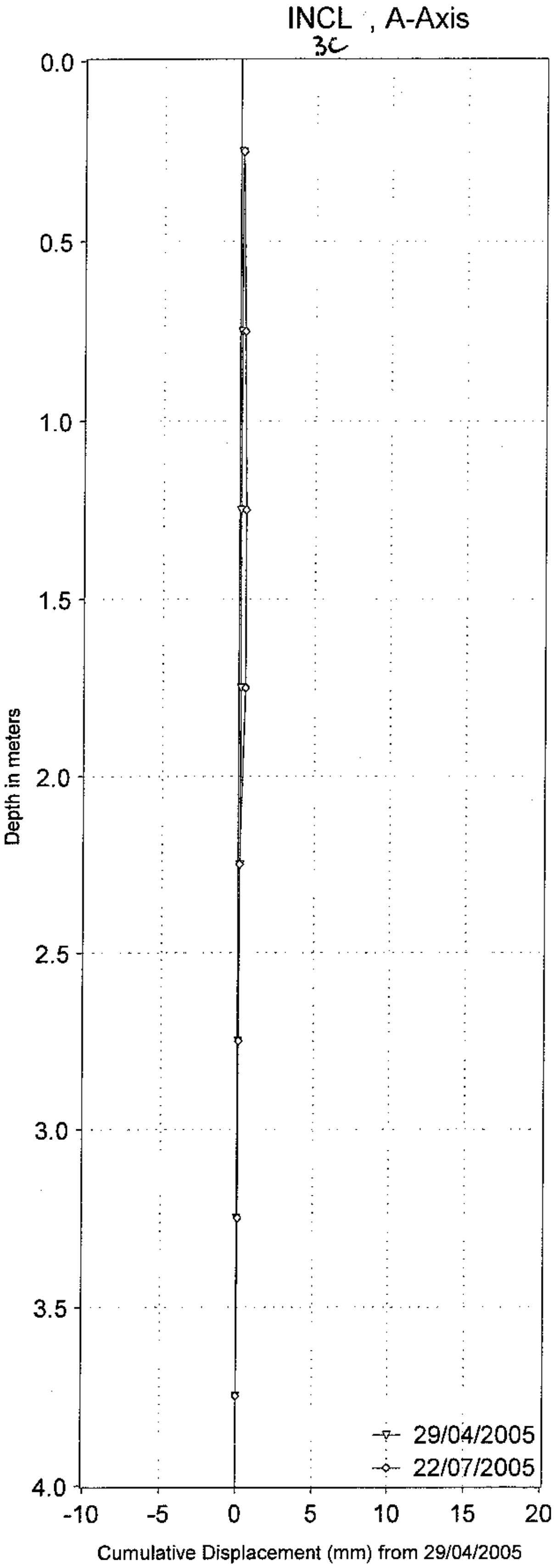


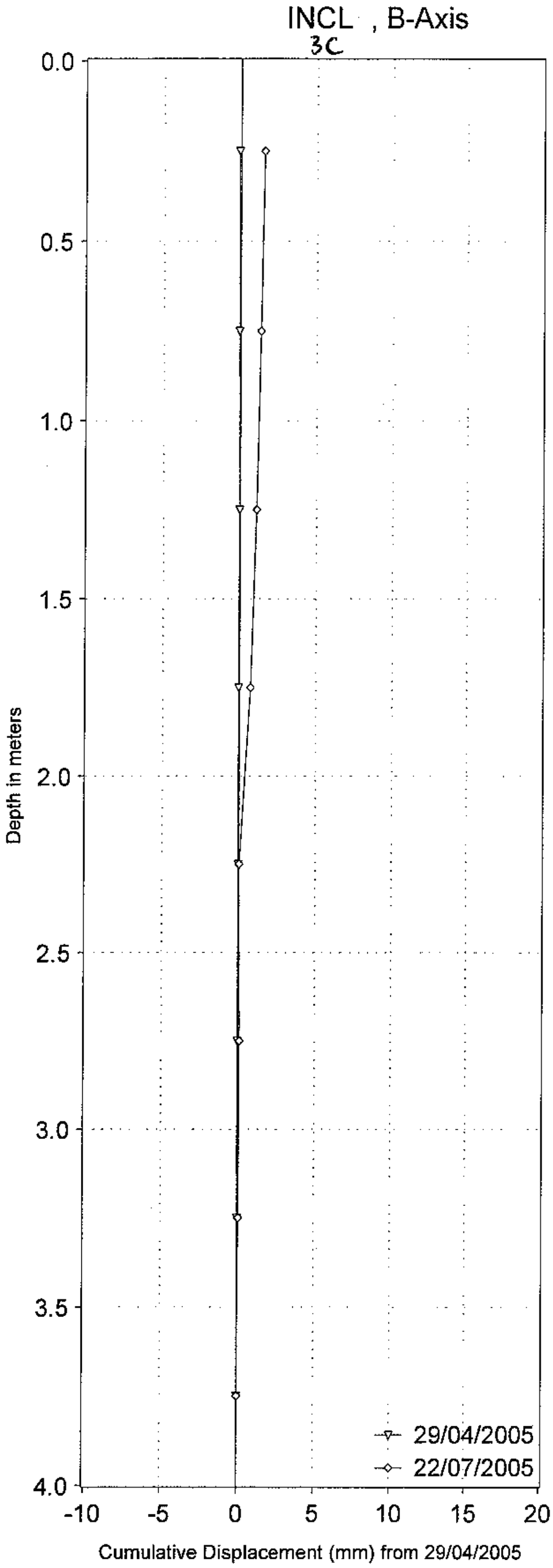


Jeffery and Katauskas Pty Ltd
CLIENT: Water Research Laboratory UNSW
PROJECT: Inclinometer Monitoring
LOCATION: Pilot Hill, Yamba, NSW

Job No: 19314WL
BH/INCL No: INCL 2C

Figure No:6





Jeffery and Katauskas Pty Ltd

CLIENT: Water Research Laboratory UNSW

PROJECT: Inclinometer Monitoring

LOCATION: Pilot Hill, Yamba, NSW

Job No: 19314WL

BH/INCL No: INCL 3C

Figure No: 7

Jeffery and Katauskas Pty Ltd

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25 November 2005
Ref: 19314WLY Let2

Water Research Laboratory
School of Civil and Environmental Engineering
University of New South Wales
King Street
MANLY VALE NSW 2093

ATTENTION: Dr Wendy Timms

Dear Wendy

INCLINOMETER MONITORING
PILOT HILL, YAMBA, NSW

This report presents the results of the second readings taken from the inclinometers installed at Pilot Hill, Yamba. The readings were completed on the 21 November 2005 in each of the installed inclinometers with the exception of Inclinometer 2C. Inclinometer 2C was unable to be read due to the presence of a car parked over the inclinometer location. The location of each inclinometer is shown on the attached Figure 1.

Following the installation and grouting of the inclinometers between the 27th and 29th April 2005, baseline readings were taken no earlier than 12 hours following grouting. These baseline readings form the reference point from which all subsequent readings are measured. Consequently, all measured movement will be in relation to the state of the hillside at the time the baseline readings were recorded.

The results of the readings taken on the 21 November 2005 when compared with those taken on the 22 July 2005 and the baseline readings indicate that at this stage no significant movement has occurred. Some movements were recorded however these were in the order of millimetres and are most likely due to 'settling in' of the casing.

The attached Figures 2 to 7 present these results. The next set of inclinometer readings is planned for the end of April 2006.

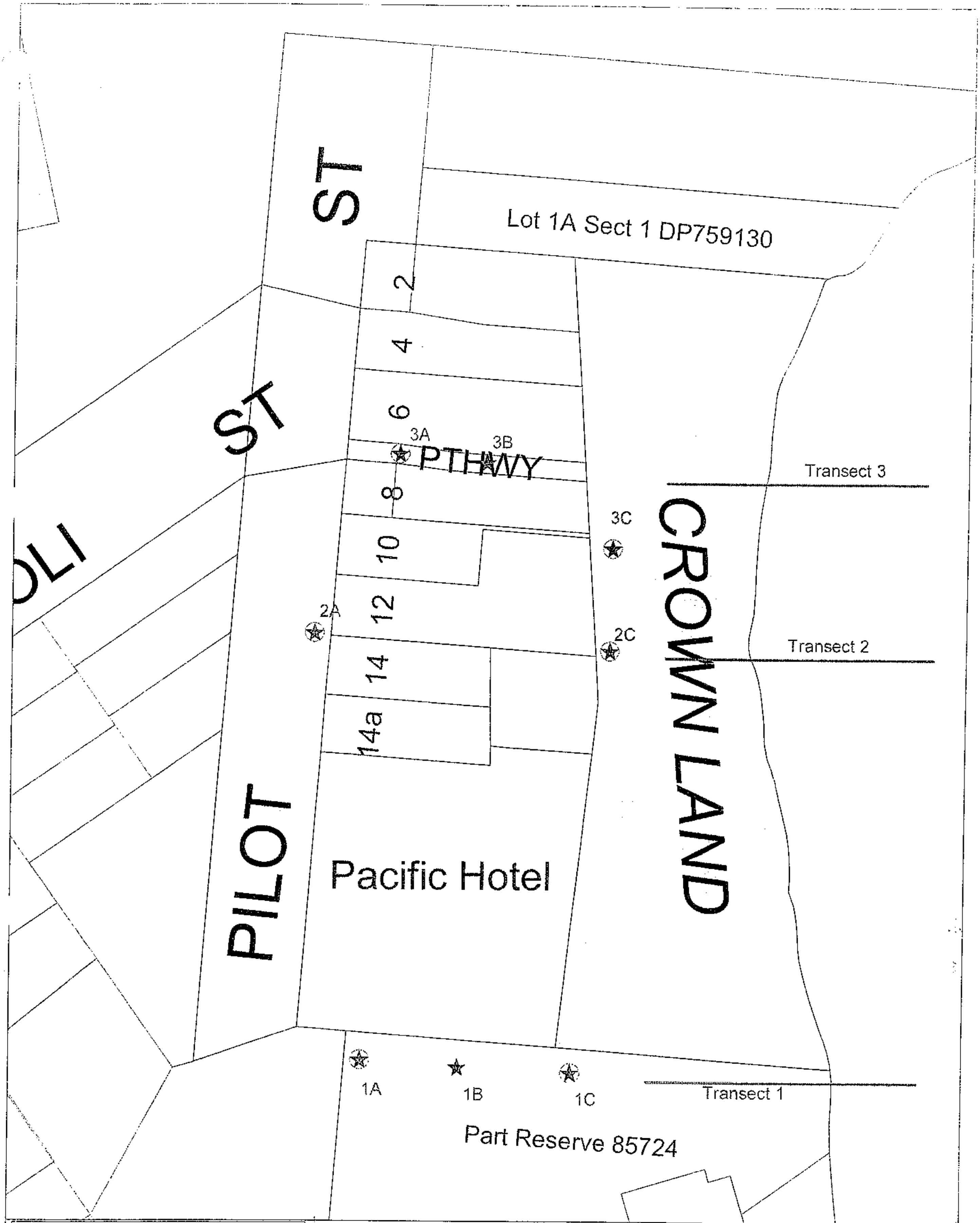
Should you require any further information regarding the above please do not hesitate to contact the undersigned.

Yours faithfully
For and on behalf of
JEFFERY AND KATAUSKAS PTY LTD

W Theunissen
Senior Geotechnical Engineer

L Speechley
Senior Associate.

Enclosed: Figure 1: Inclinometer Location Plan
Figures 2 to 6: Inclinometer Test Results

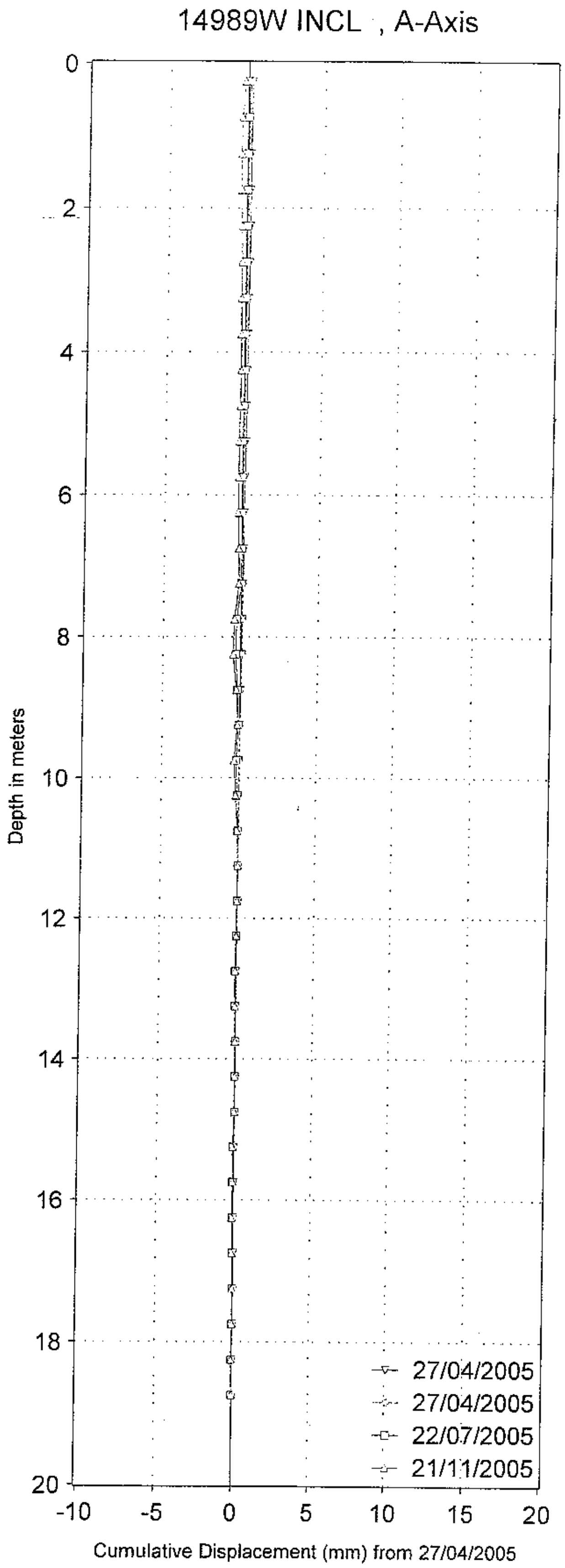


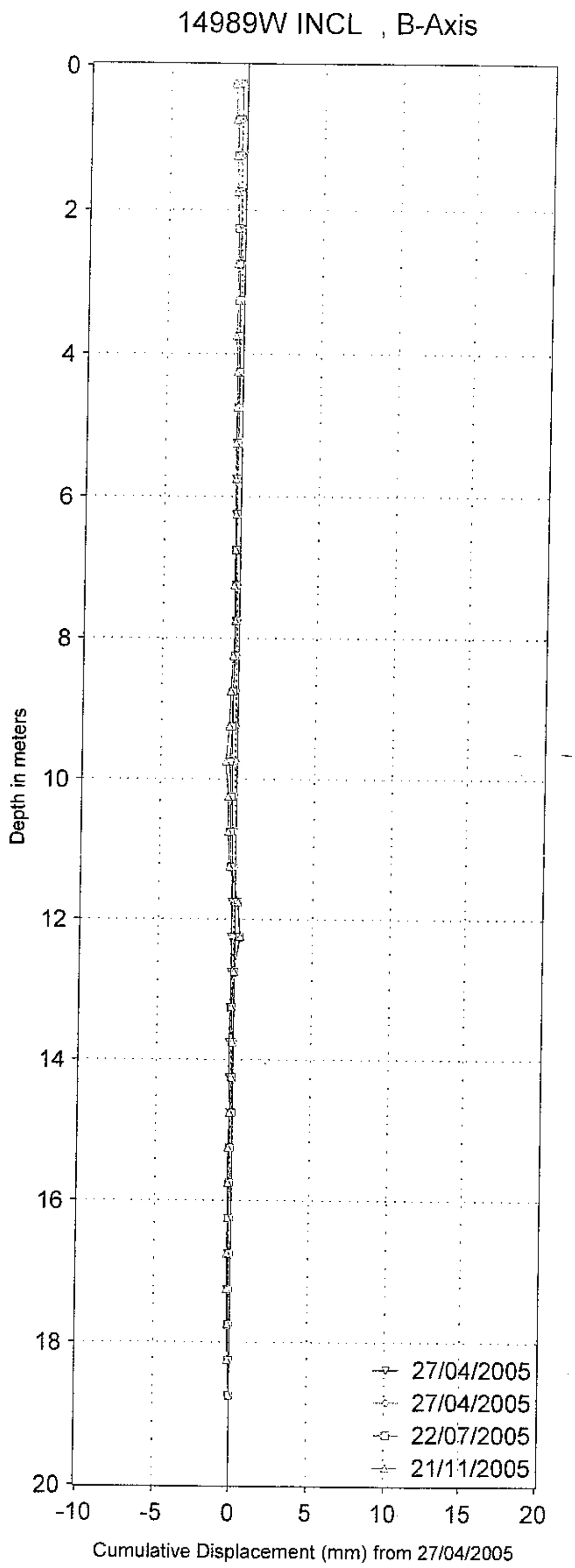
LEGEND
○ INCLINOMATER
☆ PIEZOMETER

MONITORING LOCATION PLAN

Jeffery and Katauskas Pty Ltd

Report No.19314WL Figure No.1

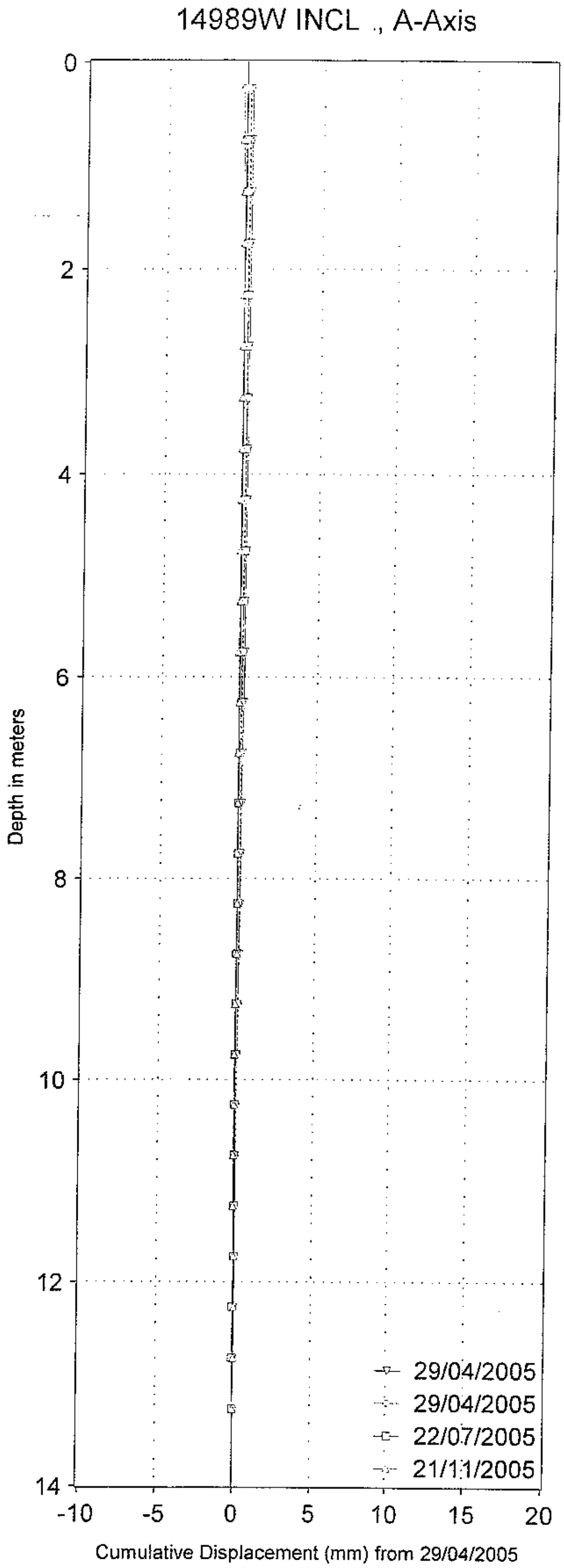


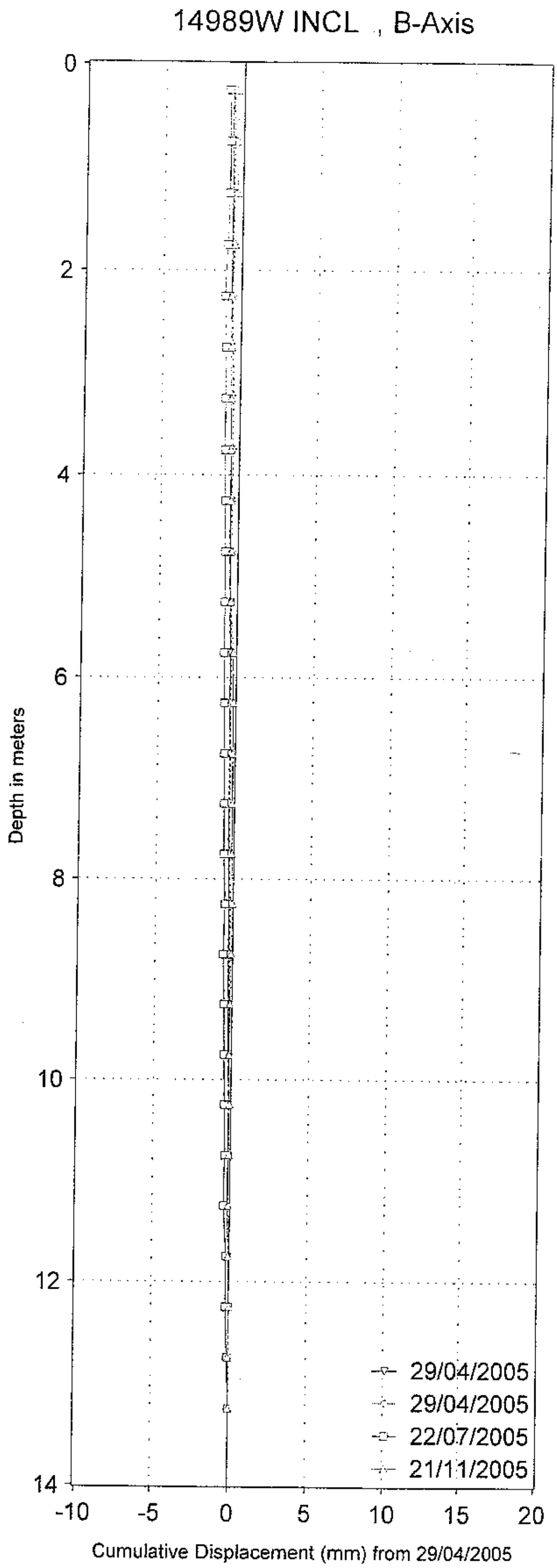


Jeffery and Katauskas Pty Ltd
CLIENT: Water Research Laboratory UNSW
PROJECT: Inclinometer Monitoring
LOCATION: Pilot Hill, Yamba, NSW

Job No: 19314WL
BH/INCL No: INCL1A

Figure No:2

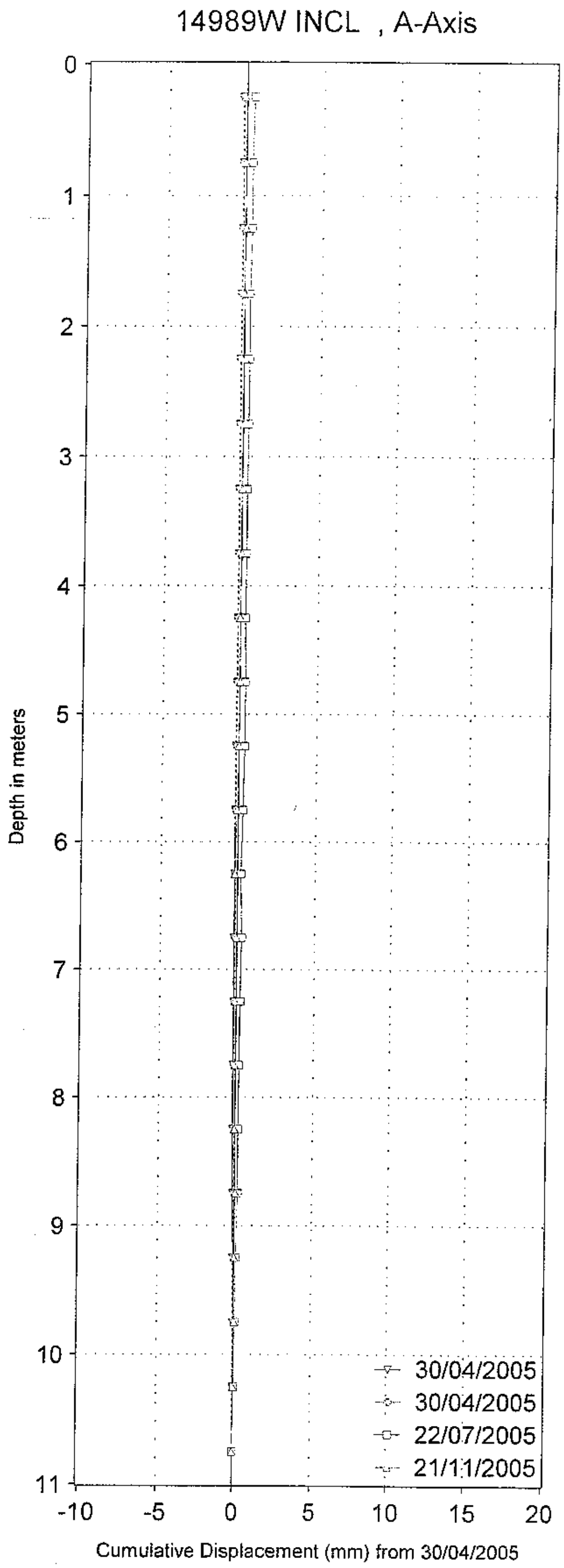


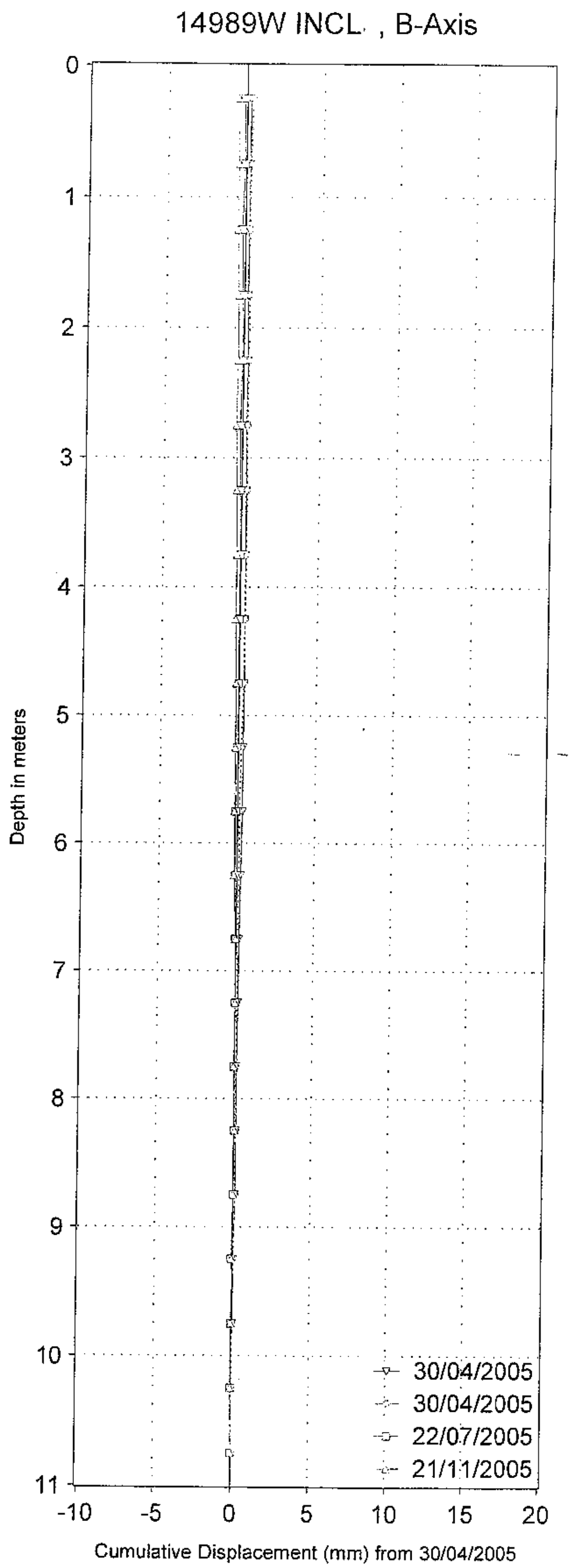


Jeffery and Katauskas Pty Ltd
CLIENT: Water Research Laboratory UNSW
PROJECT: Inclinometer Monitoring
LOCATION: Pilot Hill, Yamba, NSW

Job No: 19314WL
BH/INCL No: INCL2A

Figure No:3

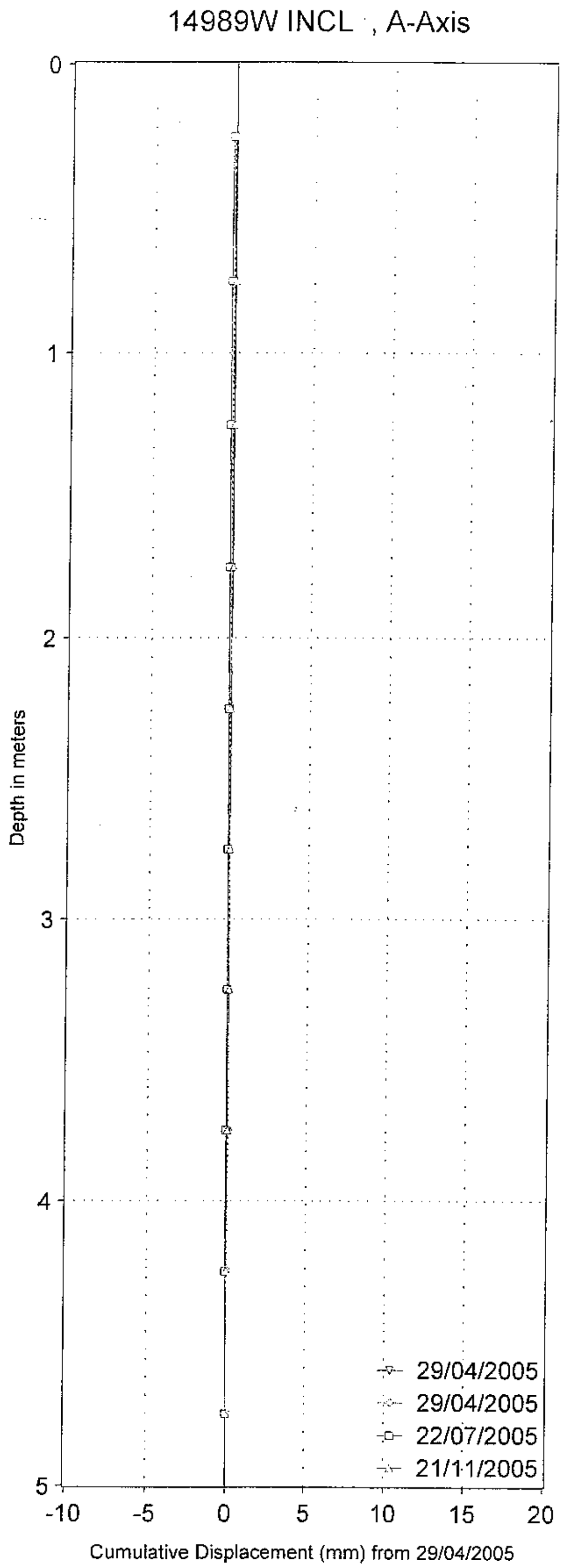


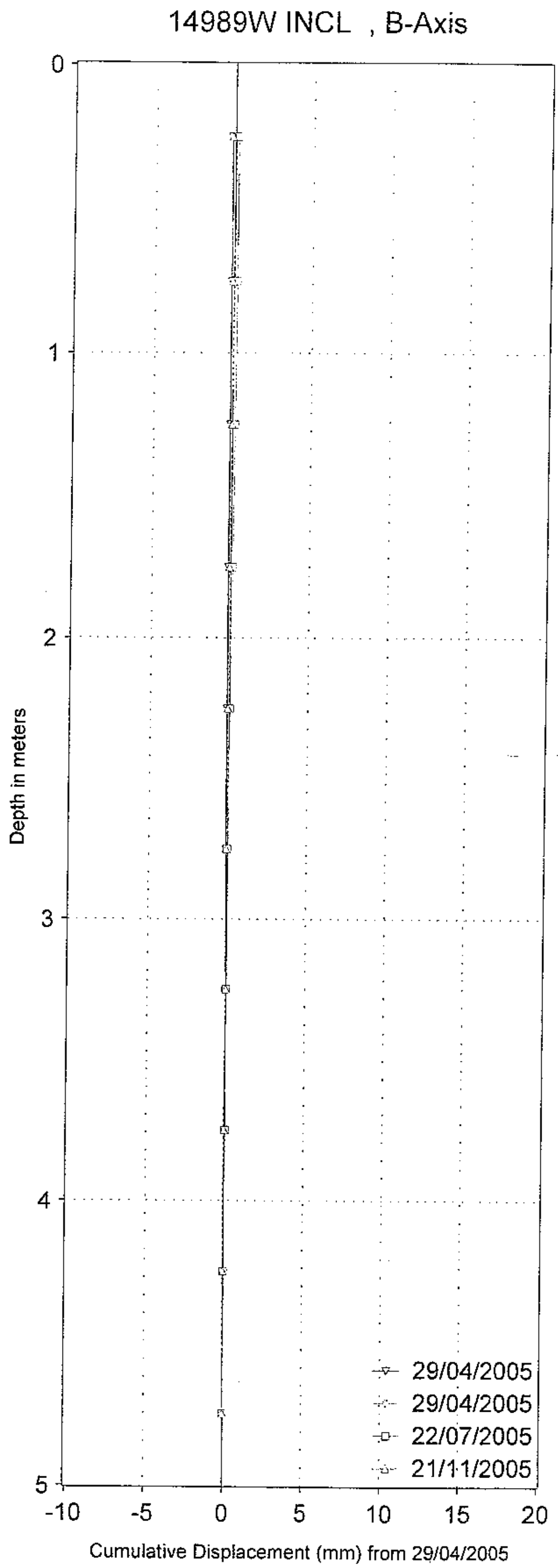


Jeffery and Katauskas Pty Ltd
CLIENT: Water Research Laboratory UNSW
PROJECT: Inclinometer Monitoring
LOCATION: Pilot Hill, Yamba, NSW

Job No: 19314WL
BH/INCL No: INCL3A

Figure No:4

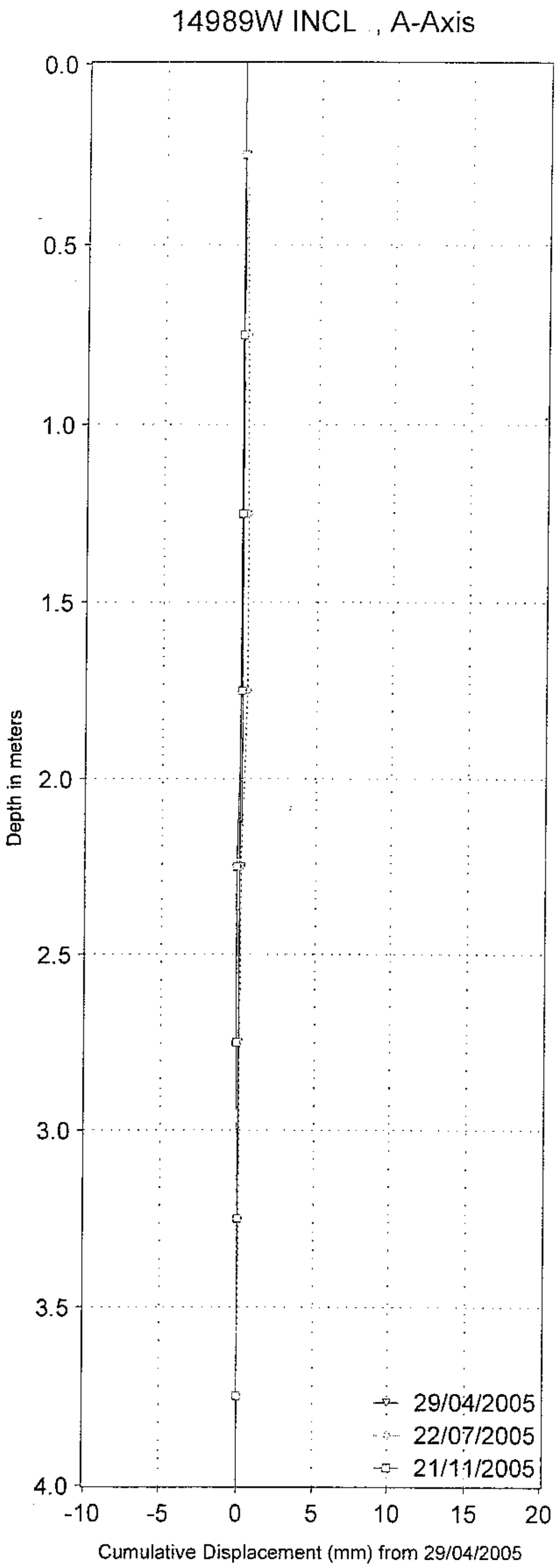


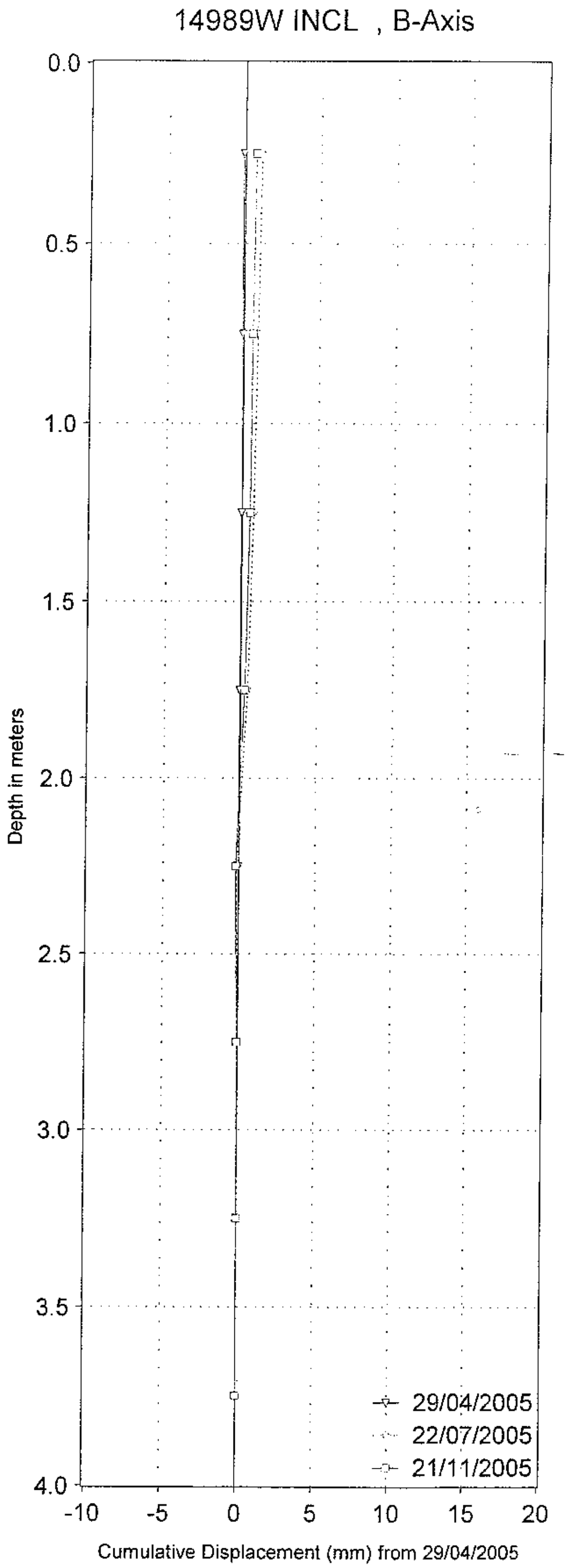


Jeffery and Katauskas Pty Ltd
CLIENT: Water Research Laboratory UNSW
PROJECT: Inclinometer Monitoring
LOCATION: Pilot Hill, Yamba, NSW

Job No: 19314WL
BH/INCL No: INCL1C

Figure No:5





Jeffery and Katauskas Pty Ltd

CLIENT: Water Research Laboratory UNSW

PROJECT: Inclinometer Monitoring

LOCATION: Pilot Hill, Yamba, NSW

Job No: 19314WL

BH/INCL No: INCL3C

Figure No:6

Jeffery and Katauskas Pty Ltd
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A ZENON BSc(Eng) GDE
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4 September 2006
Ref: 19314WLY Let3

Water Research Laboratory
School of Civil and Environmental Engineering
University of New South Wales
King Street
MANLY VALE NSW 2093

ATTENTION: Dr Wendy Timms

Dear Wendy

INCLINOMETER MONITORING
PILOT HILL, YAMBA, NSW

This report presents the results of the third readings taken from the inclinometers installed at Pilot Hill, Yamba. The readings were completed on the 17 July 2006 in each of the installed inclinometers. The location of each inclinometer is shown on the attached Figure 1.

Following the installation and grouting of the inclinometers between the 27th and 29th April 2005, baseline readings were taken no earlier than 12 hours following grouting. These baseline readings form the reference point from which all subsequent readings are measured. Consequently, all measured movement will be in relation to the state of the hillside at the time the baseline readings were recorded.

The results of the readings taken on the 17 July 2006 when compared with those taken on the 11 November 1995, 22 July 2005 and the baseline readings indicate that at this stage no significant movement has occurred. Some movements were recorded in inclinometer 2C showing signs of some downhill movement that was in the order of 2mm. This may indicate some localised creep of the soils. At this stage we do not consider this movement to be critical to warrant any immediate action.

The attached Figures 2 to 7 present these results. The next set of inclinometer readings is planned for the end of January 2006.

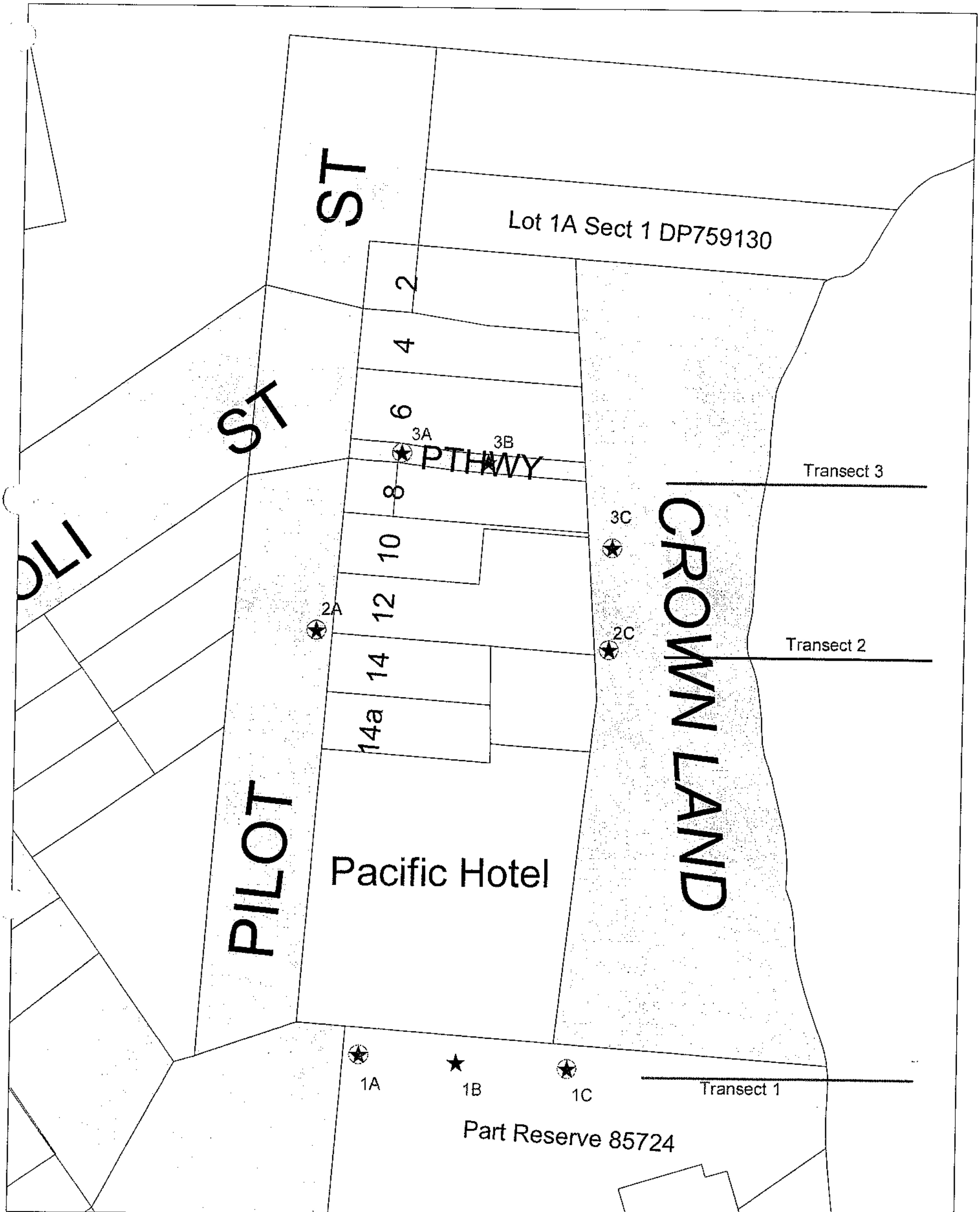
Should you require any further information regarding the above please do not hesitate to contact the undersigned.

Yours faithfully
For and on behalf of
JEFFERY AND KATAUSKAS PTY LTD

for W Theunissen
Senior Geotechnical Engineer

L Speechley
Senior Associate.

Enclosed: Figure 1: Inclinometer Location Plan
Figures 2 to 7: Inclinometer Test Results



LEGEND

○ INCLINOMATER

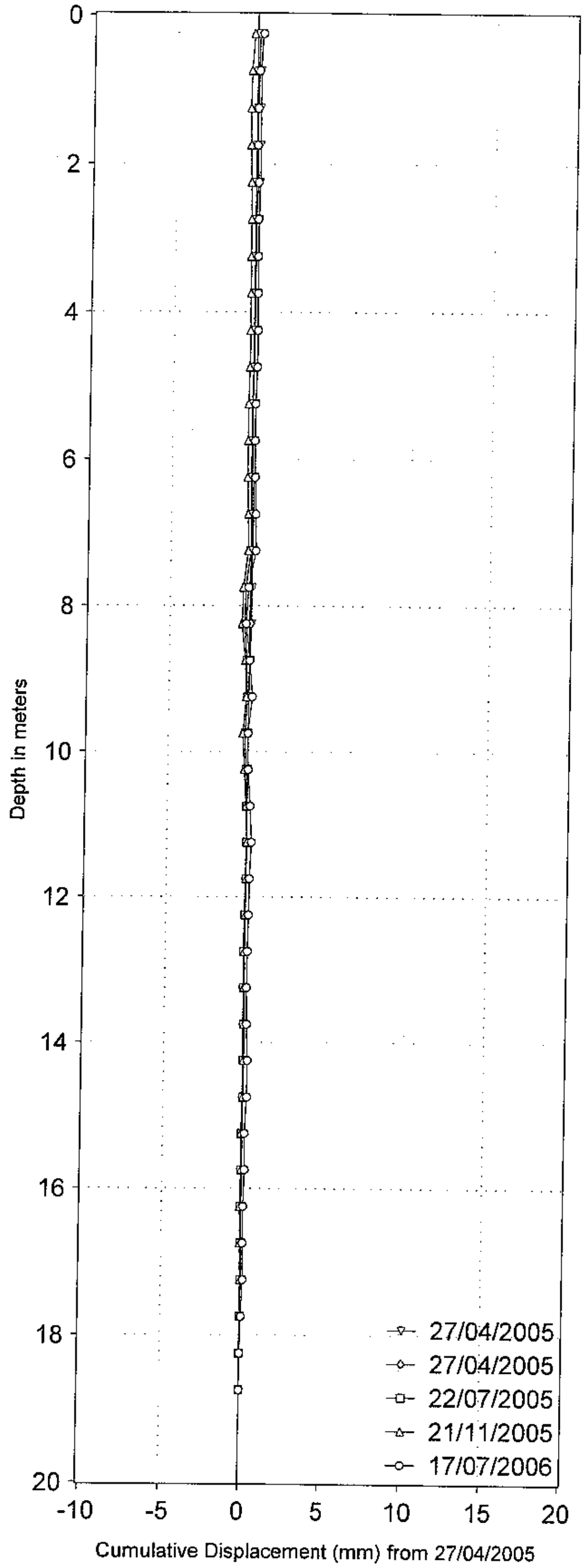
★ PIEZOMETER

MONITORING LOCATION PLAN

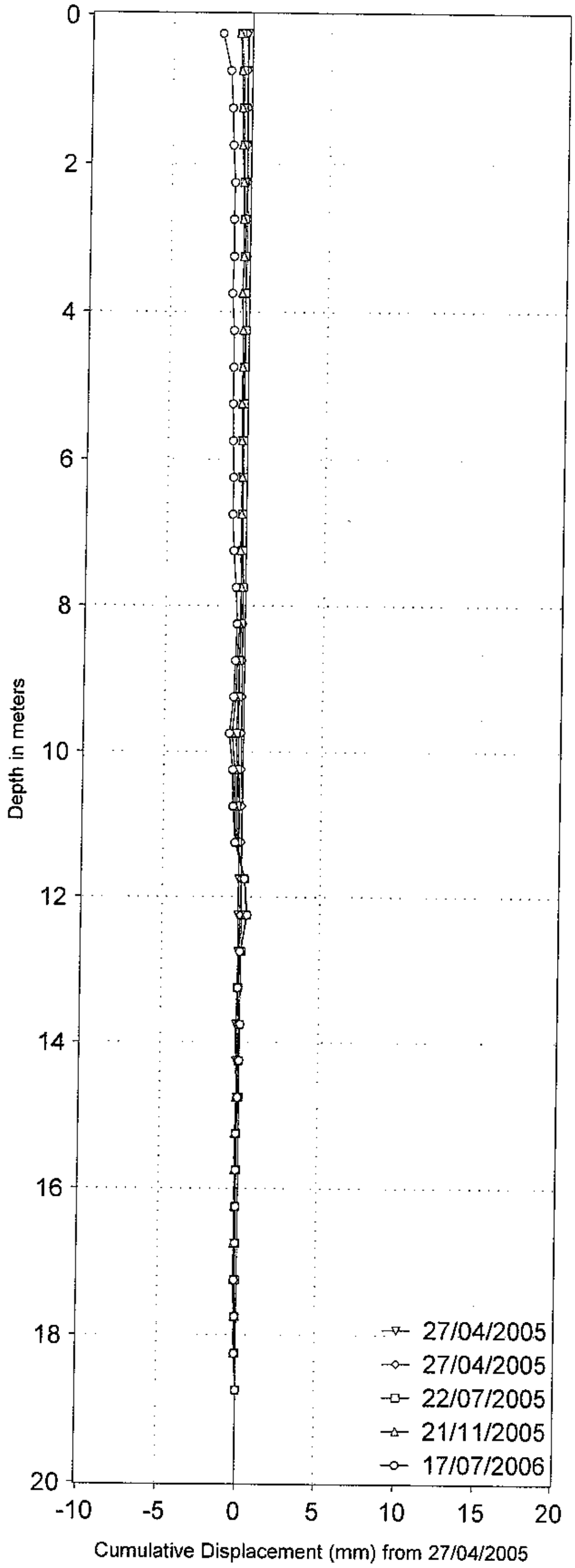
Jeffery and Katauskas Pty Ltd

Report No.19314WL Figure No.1

19314WL INCL1A, A-Axis



19314WL INCL1A, B-Axis



Jeffery and Katauskas Pty Ltd

CLIENT: Water Research Laboratory UNSW

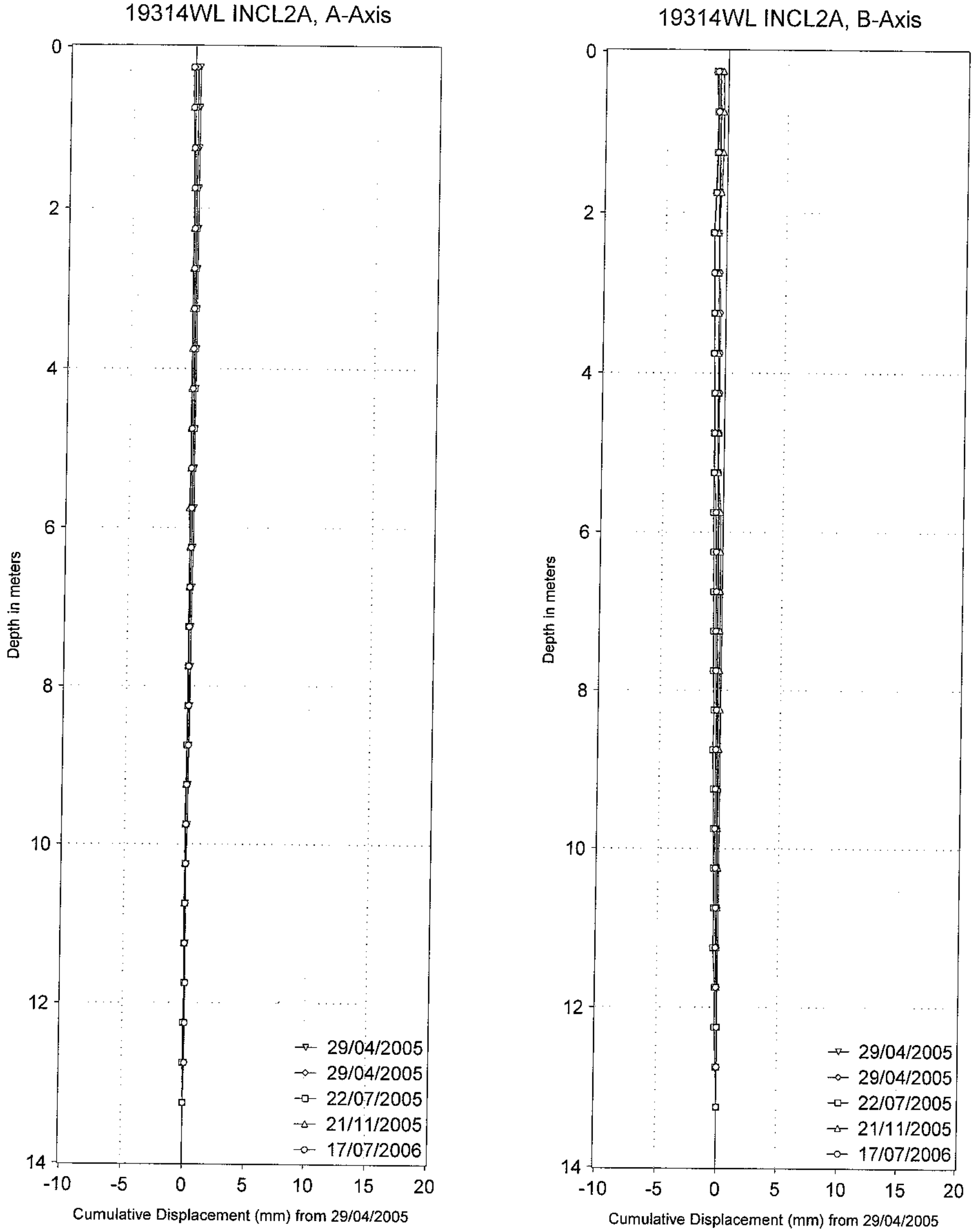
PROJECT: Inclinometer Monitoring

LOCATION: Pilot Hill, Yamba, NSW

Job No: 19314WL

BH/INCL No: INCL1A

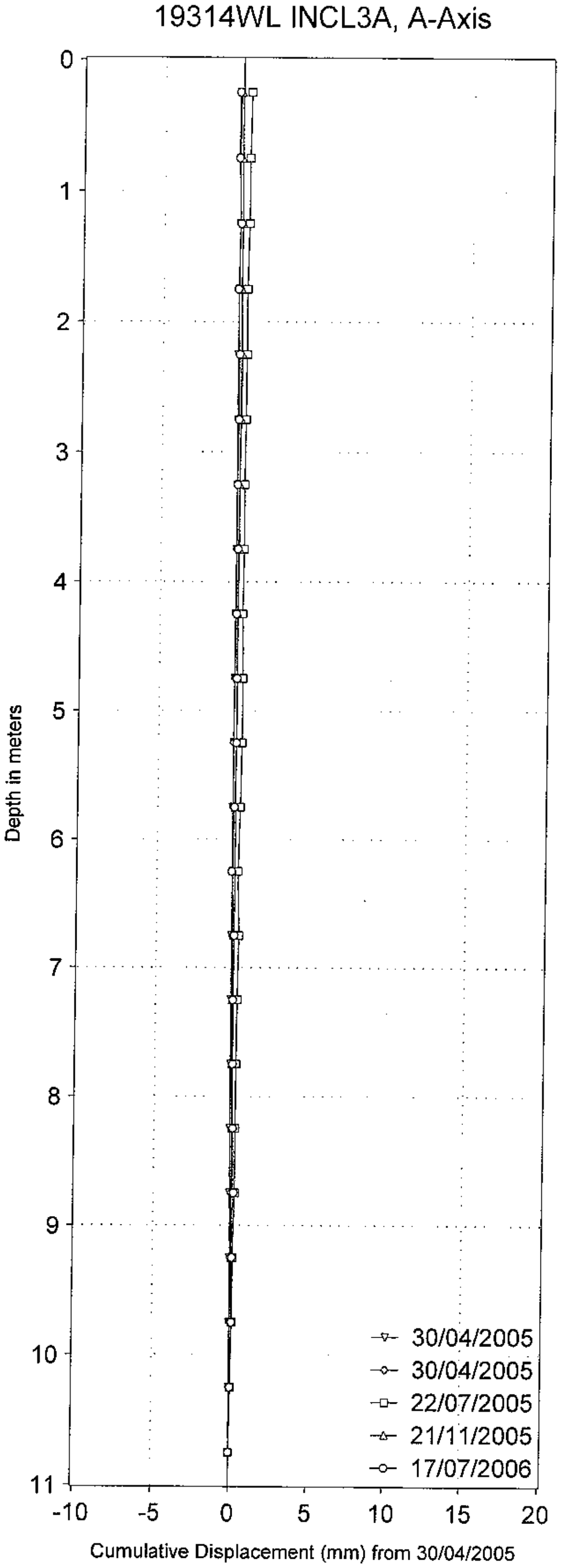
Figure No:2

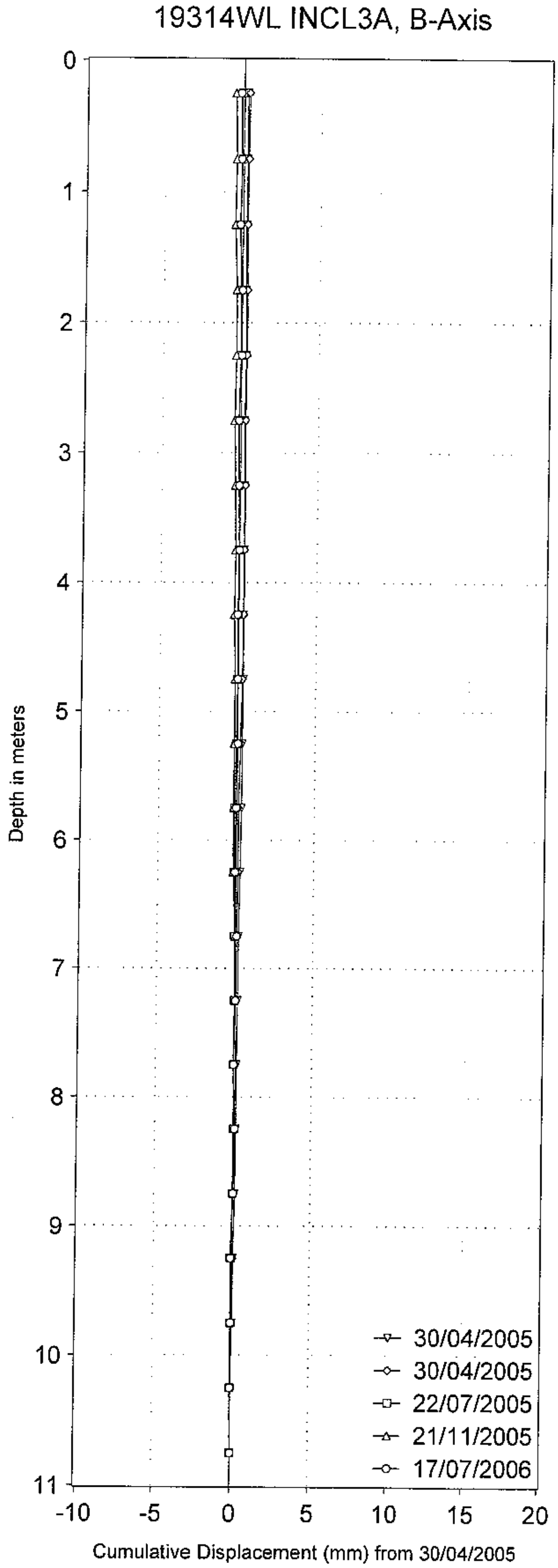


Jeffery and Katauskas Pty Ltd
CLIENT: Water Research Laboratory UNSW
PROJECT: Inclinometer Monitoring
LOCATION: Pilot Hill, Yamba, NSW

Job No: 19314WL
BH/INCL No: INCL2A

Figure No:3





Jeffery and Katauskas Pty Ltd

CLIENT: Water Research Laboratory UNSW

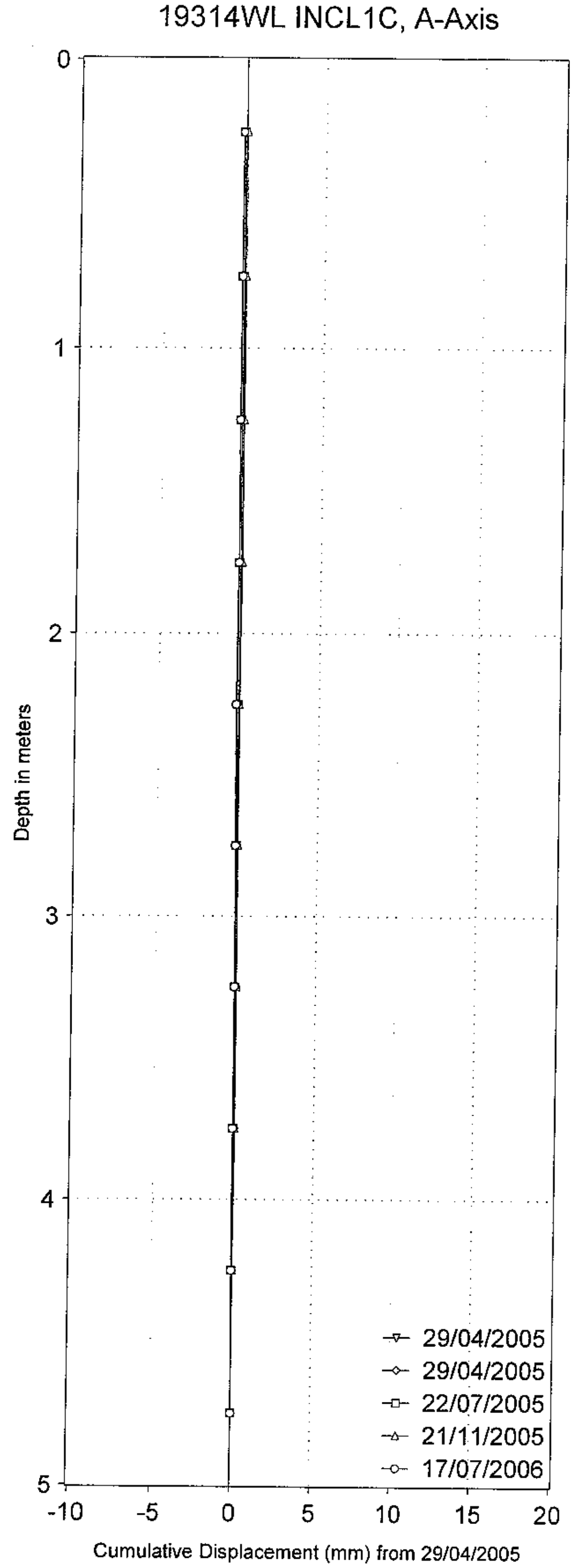
PROJECT: Inclinometer Monitoring

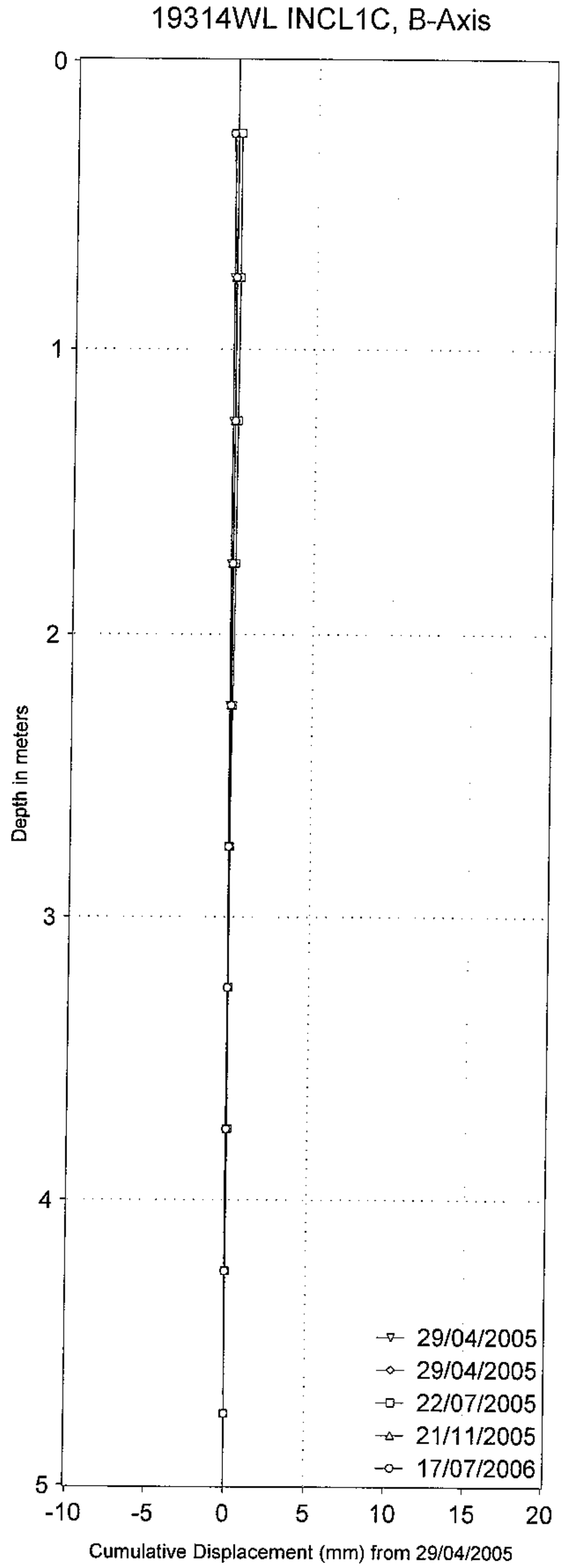
LOCATION: Pilot Hill, Yamba, NSW

Job No: 19314WL

BH/INCL No: INCL3A

Figure No:4

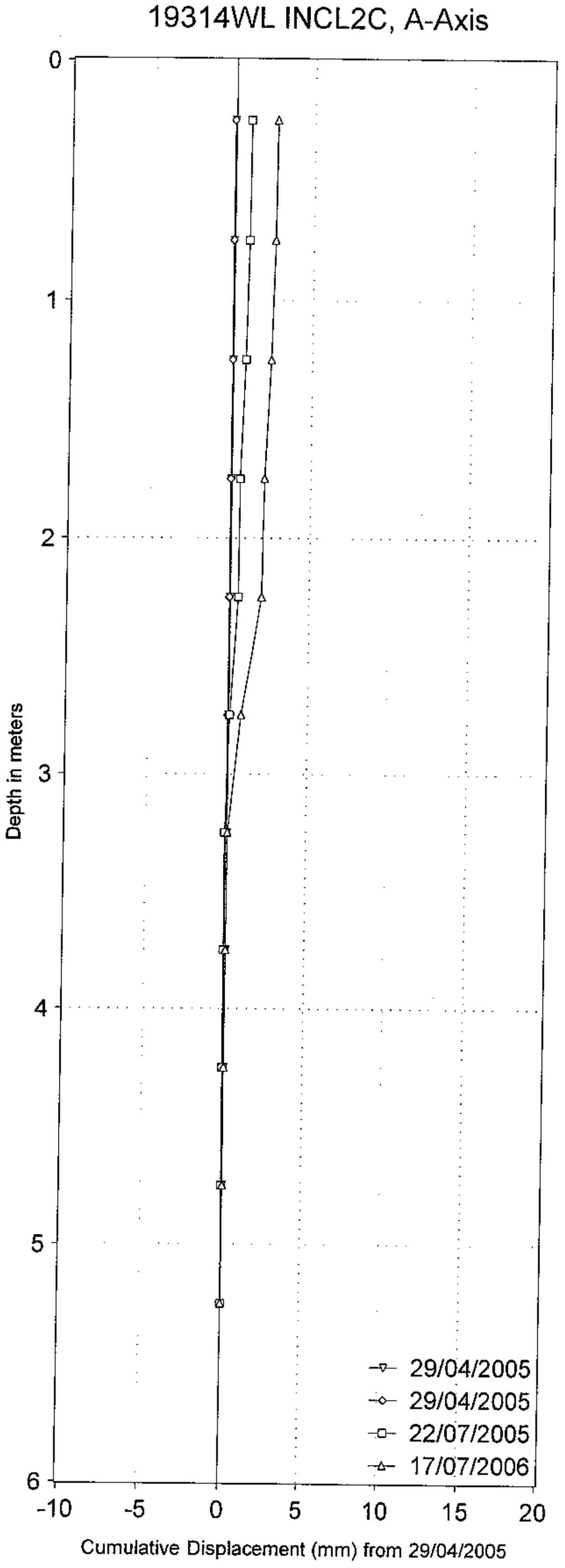


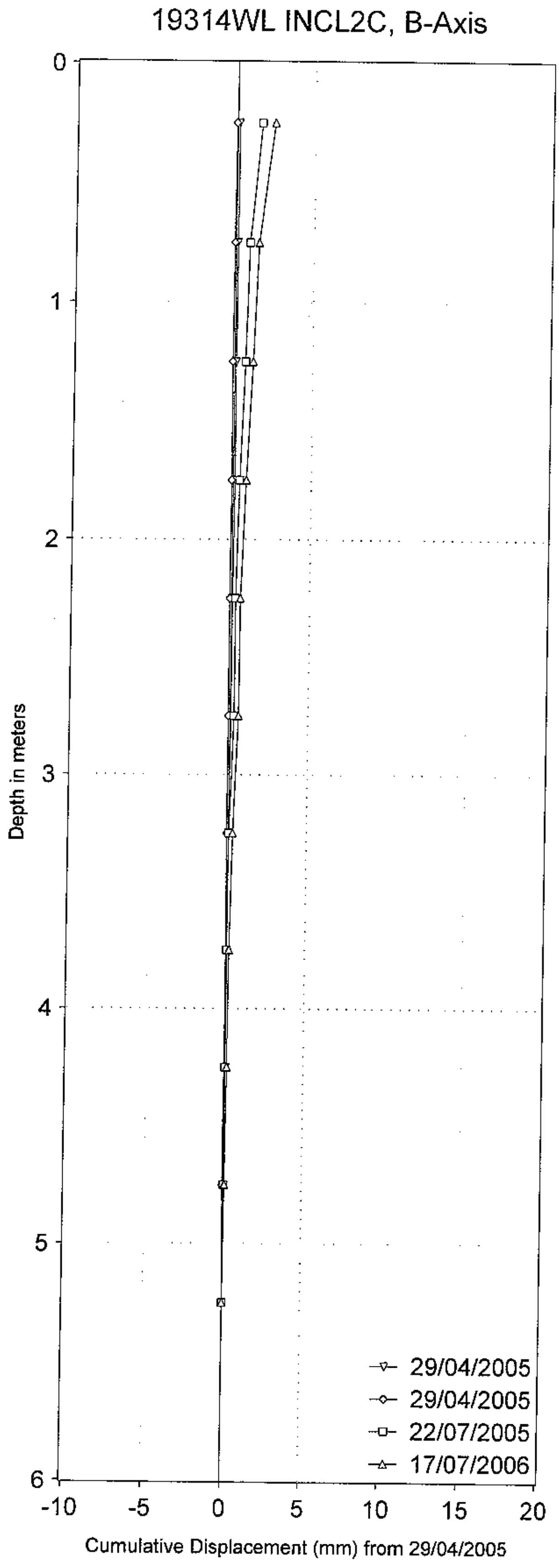


Jeffery and Katauskas Pty Ltd
CLIENT: Water Research Laboratory UNSW
PROJECT: Inclinometer Monitoring
LOCATION: Pilot Hill, Yamba, NSW

Job No: 19314WL
BH/INCL No: INCL1C

Figure No:5





Jeffery and Katauskas Pty Ltd

CLIENT: Water Research Laboratory UNSW

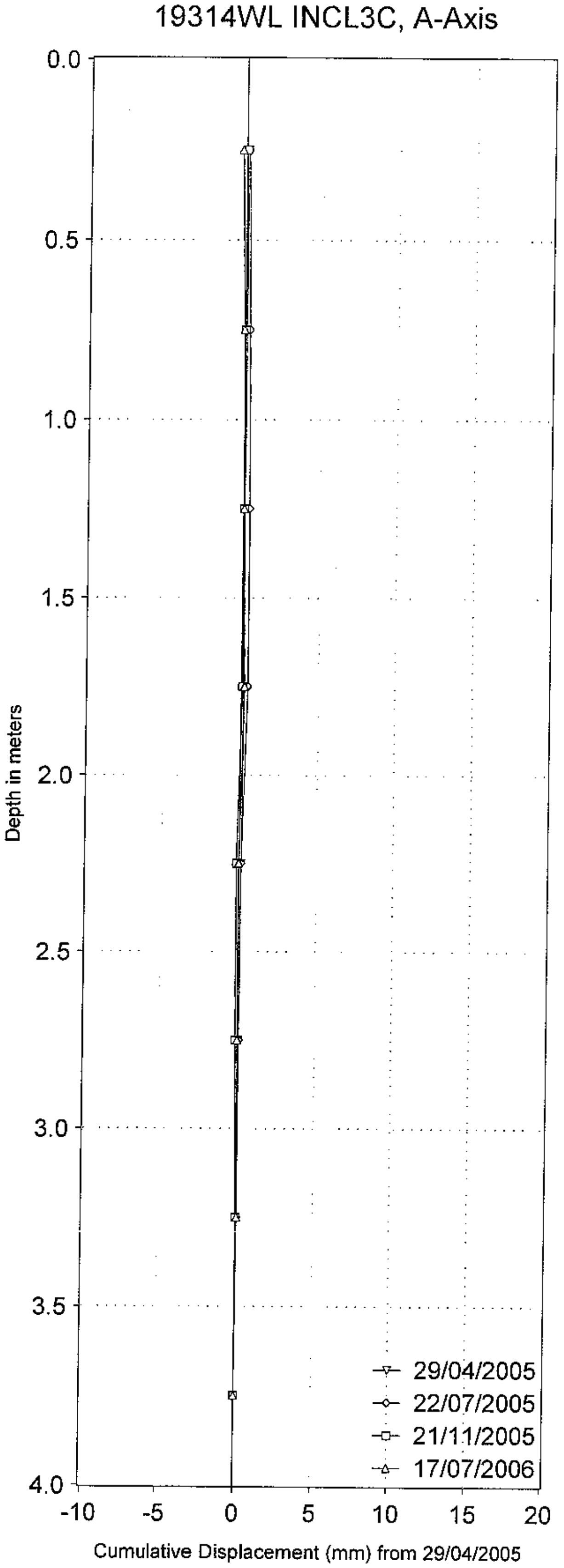
PROJECT: Inclinometer Monitoring

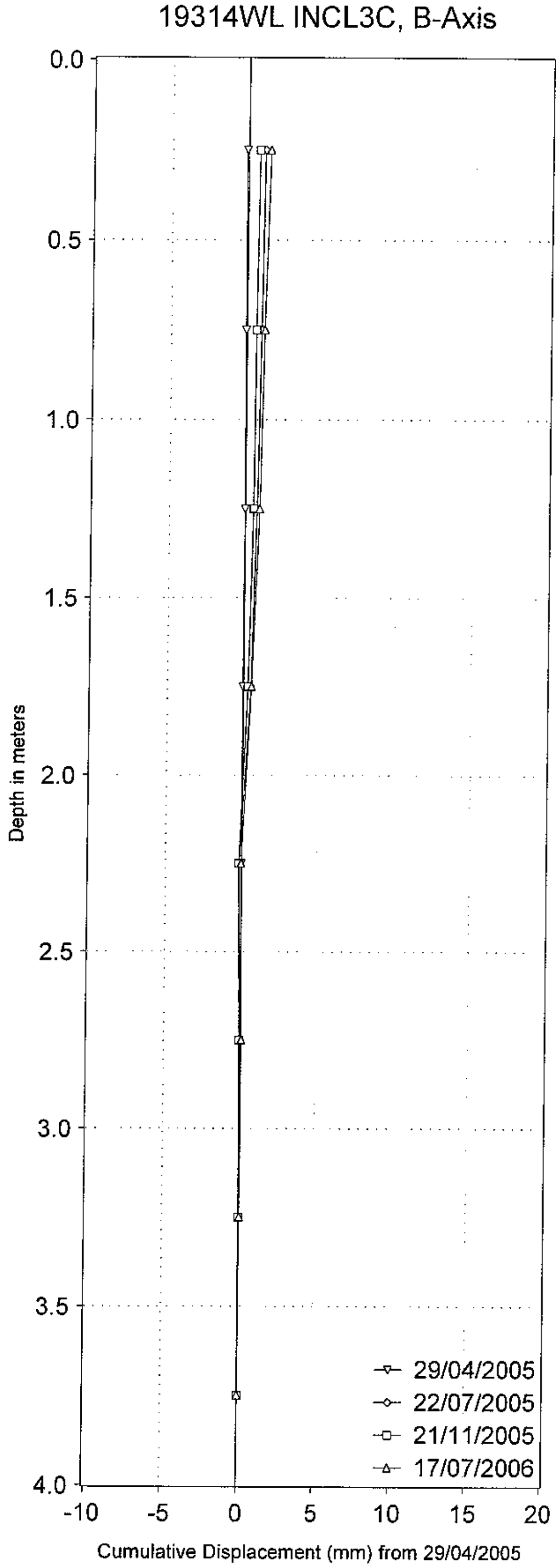
LOCATION: Pilot Hill, Yamba, NSW

Job No: 19314WL

BH/INCL No: INCL2C

Figure No:6





Jeffery and Katauskas Pty Ltd
CLIENT: Water Research Laboratory UNSW
PROJECT: Inclinometer Monitoring
LOCATION: Pilot Hill, Yamba, NSW

Job No: 19314WL
BH/INCL No: INCL3C

Figure No:7

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS
A.B.N. 17 003 550 801 A.C.N. 003 550 801

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P STUBBS BSc MICE FGS
D TREWEEK Dip Tech
E H FLETCHER BSc (Eng) ME

Senior Associates
F A VEGA BSc(Eng) GDE
A ZENON BSc(Eng) GDE
P C WRIGHT BE(Hons) MEngSc
L J SPEECHLEY BE(Hons) MEngSc

Associates
A B WALKER BE(Hons) MEngSc

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Tel: 02-9809 7322
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Fax: 02-9809 7626

2 April 2007
Ref: 19314WLYLet4

Water Research Laboratory
School of Civil and Environmental Engineering
University of New South Wales
King Street
MANLY VALE NSW 2093

ATTENTION: Dr Wendy Timms

Dear Wendy

INCLINOMETER MONITORING
PILOT HILL, YAMBA, NSW

This report presents the results of the fourth readings taken from the inclinometers installed at Pilot Hill, Yamba. The readings were completed on 8 March 2007 in each of the installed inclinometers. The location of each inclinometer is shown on the attached Figure 1.

Following the installation and grouting of the inclinometers between 27 and 29 April 2005, baseline readings were taken no earlier than 12 hours following grouting. These baseline readings form the reference point from which all subsequent readings are measured. Consequently, all measured movement will be in relation to the state of the hillside at the time the baseline readings were recorded.

The results of the readings taken on 8 March 2007 when compared with those taken on 17 July 2006, 21 November 2005 and 22 July 2005 and the baseline readings indicate that at this stage negligible movement has occurred in all the inclinometers with the exception of Inclinometer 2C. Inclinometer 2C is showing signs of some downhill movement in the order of 5mm. This movement is probably indicative of localised creep of the soils. At this stage we do not consider this movement to be critical to warrant any immediate action, however further monitoring is recommended and if the trend continues then action may be required.

The attached Figures 2 to 7 present these results. We suggest the next set of inclinometer readings be carried out in about August 2007, which is about 3 months after the planned final completion of the monitoring.

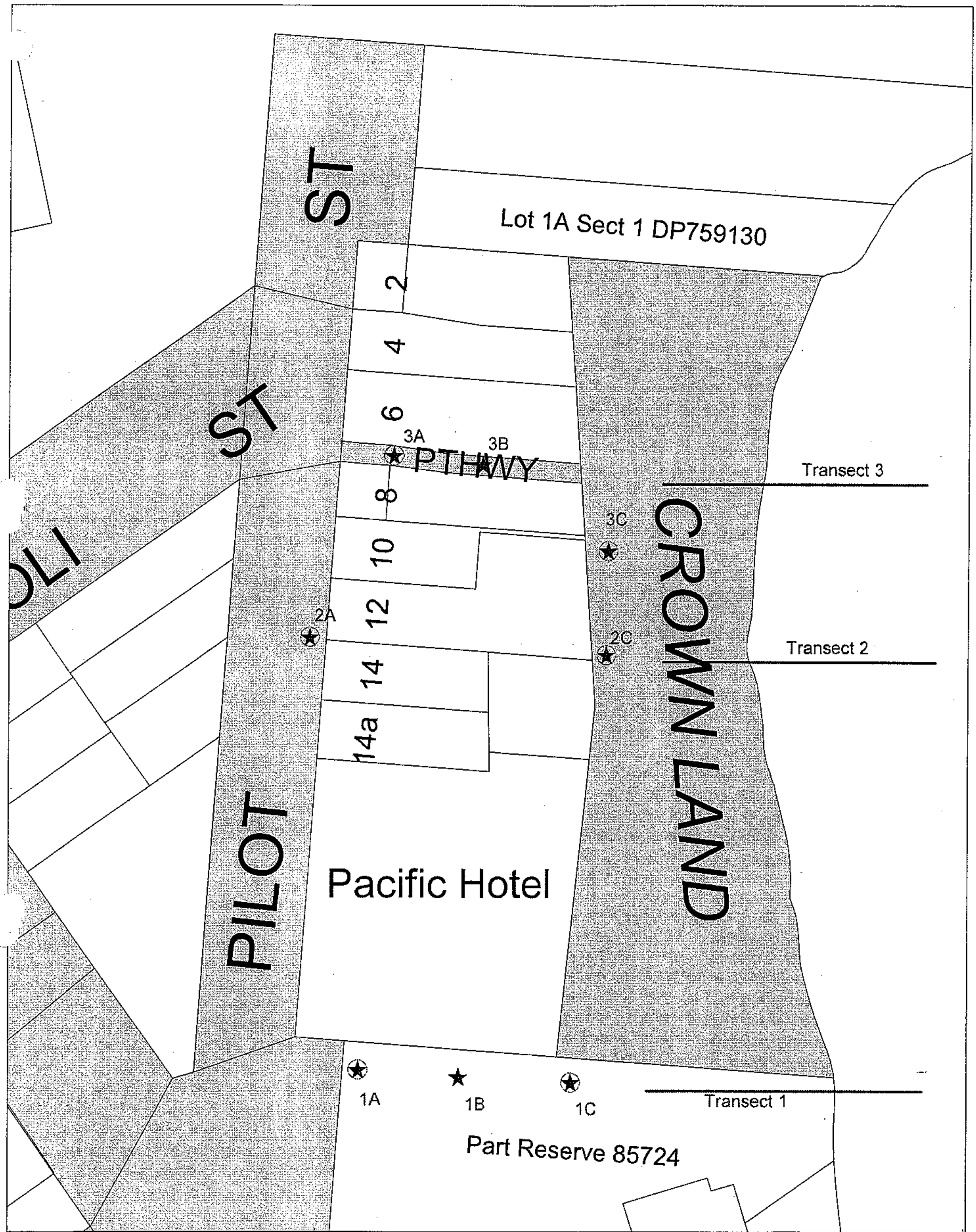
Should you require any further information regarding the above please do not hesitate to contact the undersigned.

Yours faithfully
For and on behalf of
JEFFERY AND KATAUSKAS PTY LTD

L Speechley

L Speechley
Senior Associate.

Enclosed: Figure 1: Inclinometer Location Plan
Figures 2 to 7: Inclinometer Test Results

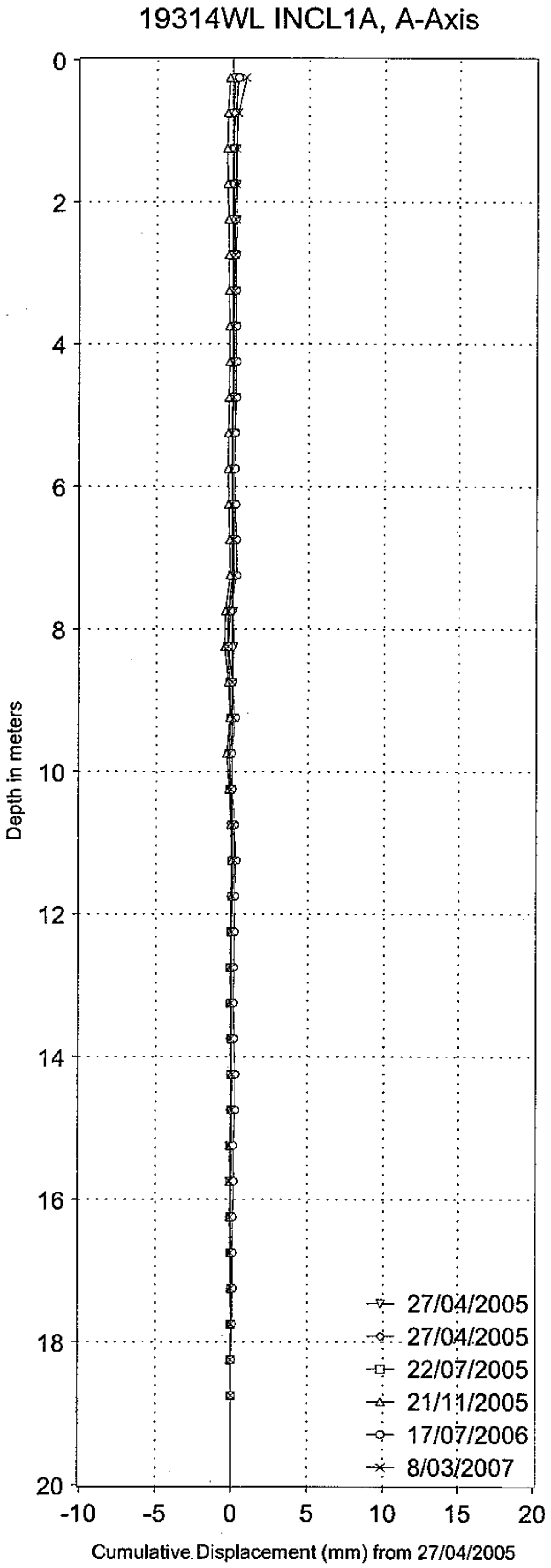


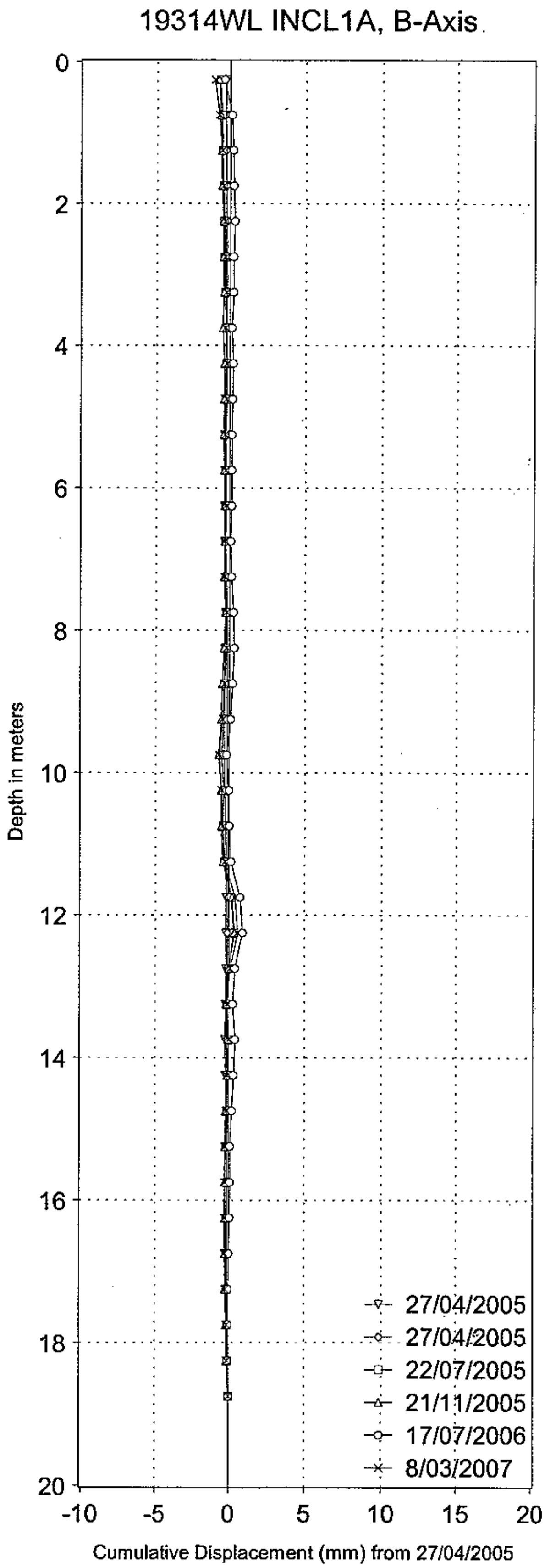
LEGEND
○ INCLINOMATER
★ PIEZOMETER

MONITORING LOCATION PLAN

Jeffery and Katauskas Pty Ltd

Report No. 19314WL   Figure No. 1

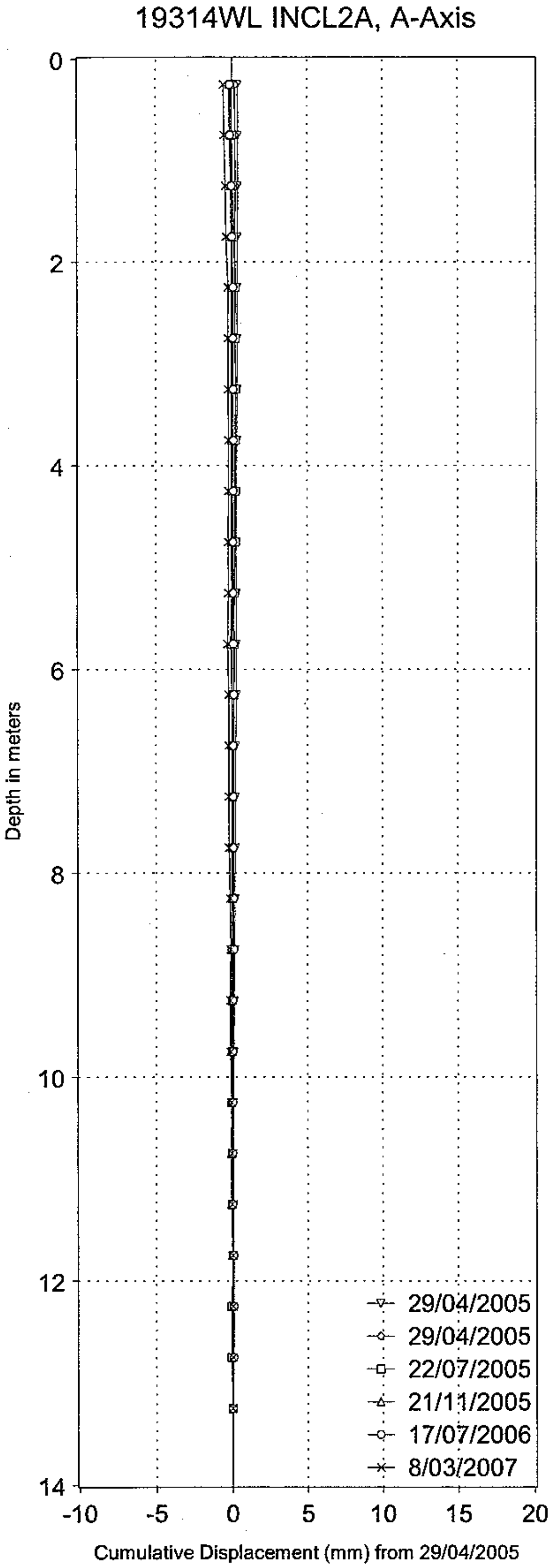


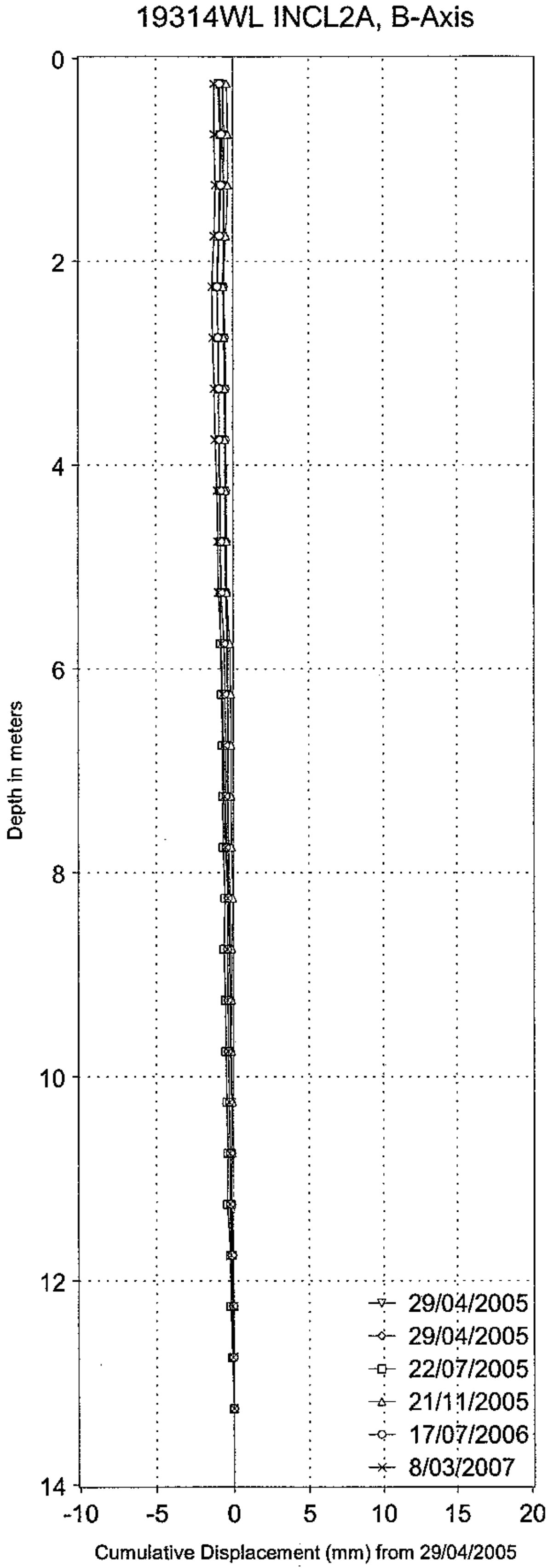


Jeffery and Katauskas Pty Ltd
CLIENT: Water Research Laboratory UNSW
PROJECT: Inclinometer Monitoring
LOCATION: Pilot Hill, Yamba, NSW

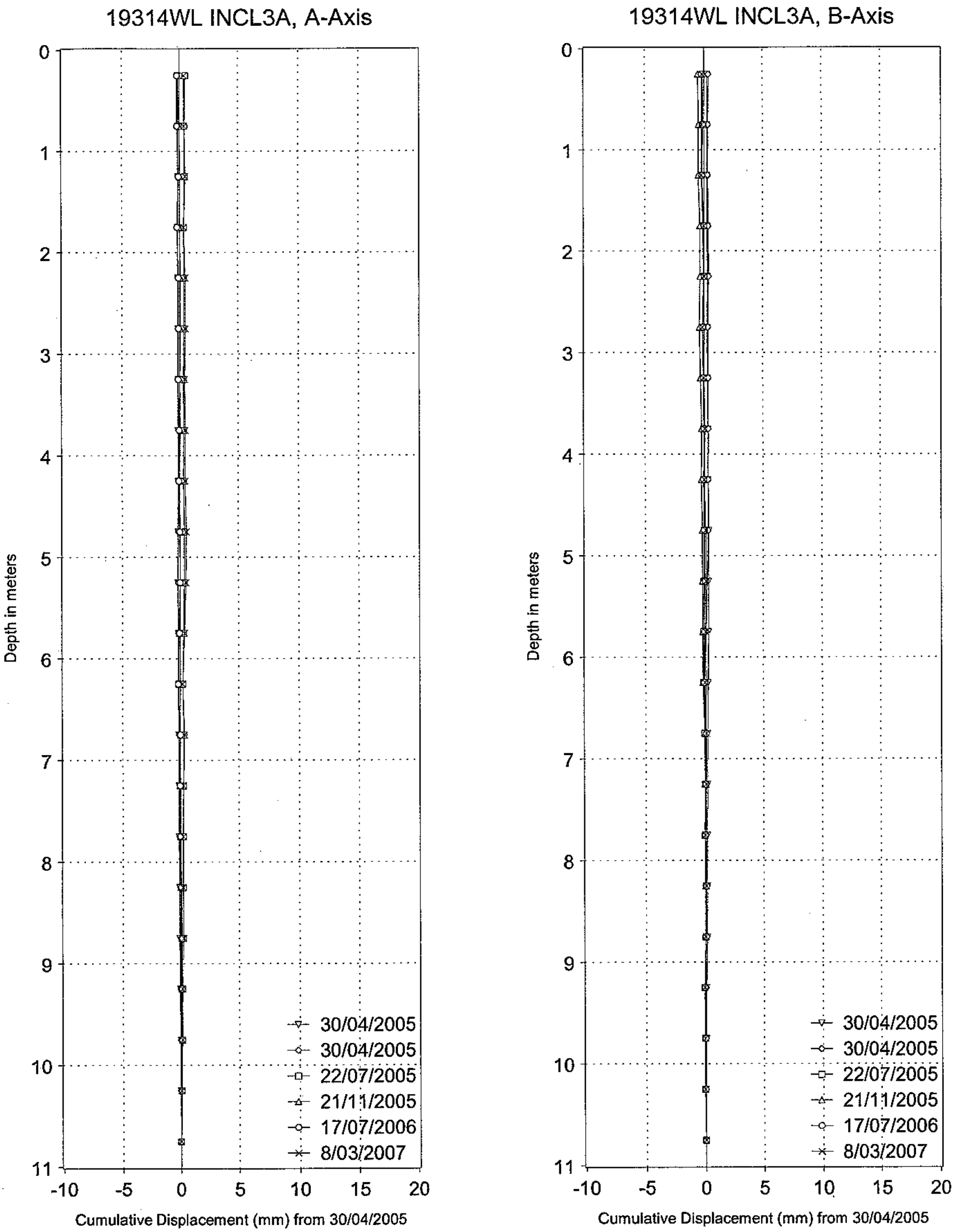
Job No: 19314WL
BH/INCL No: INCL1A

Figure No: 2





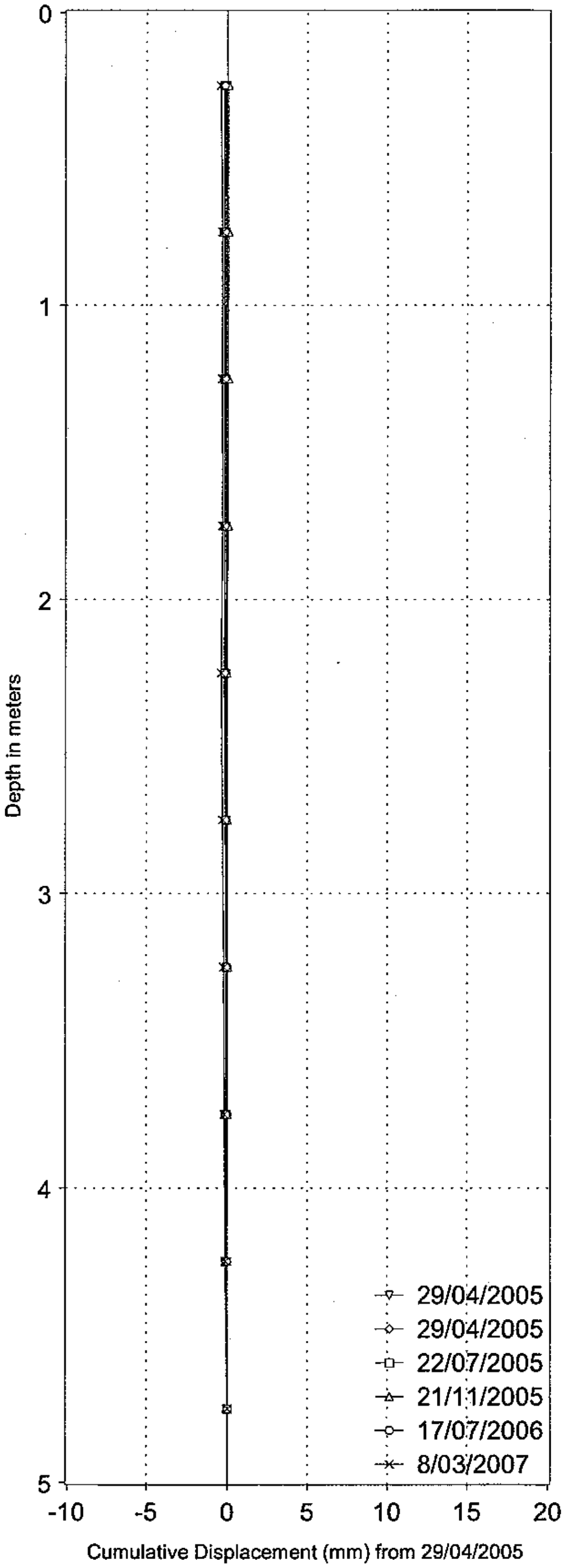
Jeffery and Katauskas Pty Ltd	**Job No: 19314WL**
CLIENT: Water Research Laboratory UNSW	**BH/INCL No: INCL2A**
PROJECT: Inclinometer Monitoring	
LOCATION: Pilot Hill, Yamba, NSW	**Figure No:** 3



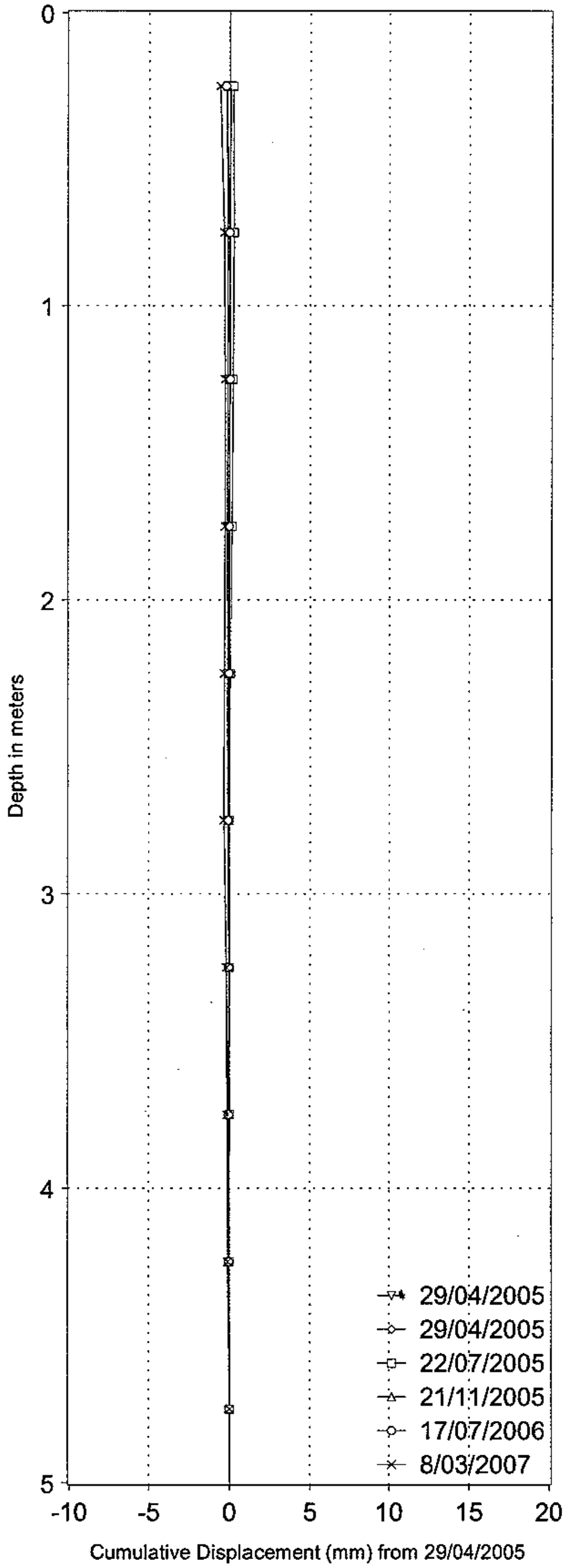
Jeffery and Katauskas Pty Ltd
CLIENT: Water Research Laboratory UNSW
PROJECT: Inclinometer Monitoring
LOCATION: Pilot Hill, Yamba, NSW

Job No: 19314WL
BH/INCL No: INCL3A
Figure No: 4

19314WL INCL1C, A-Axis



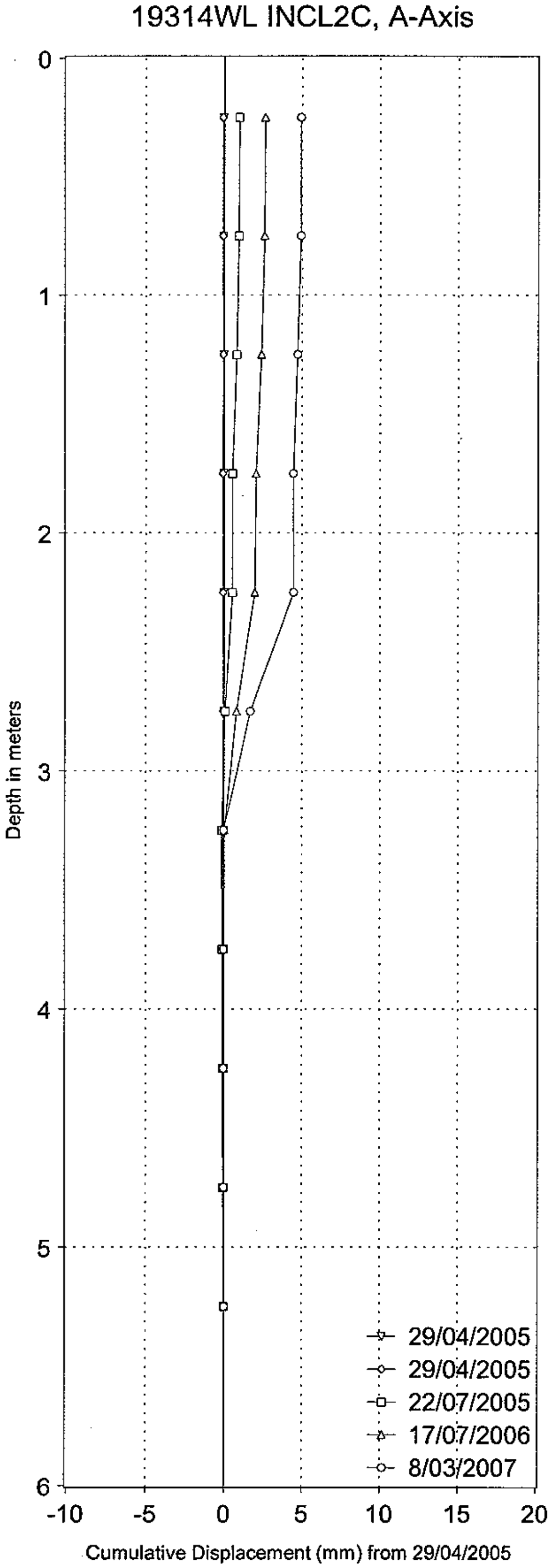
19314WL INCL1C, B-Axis

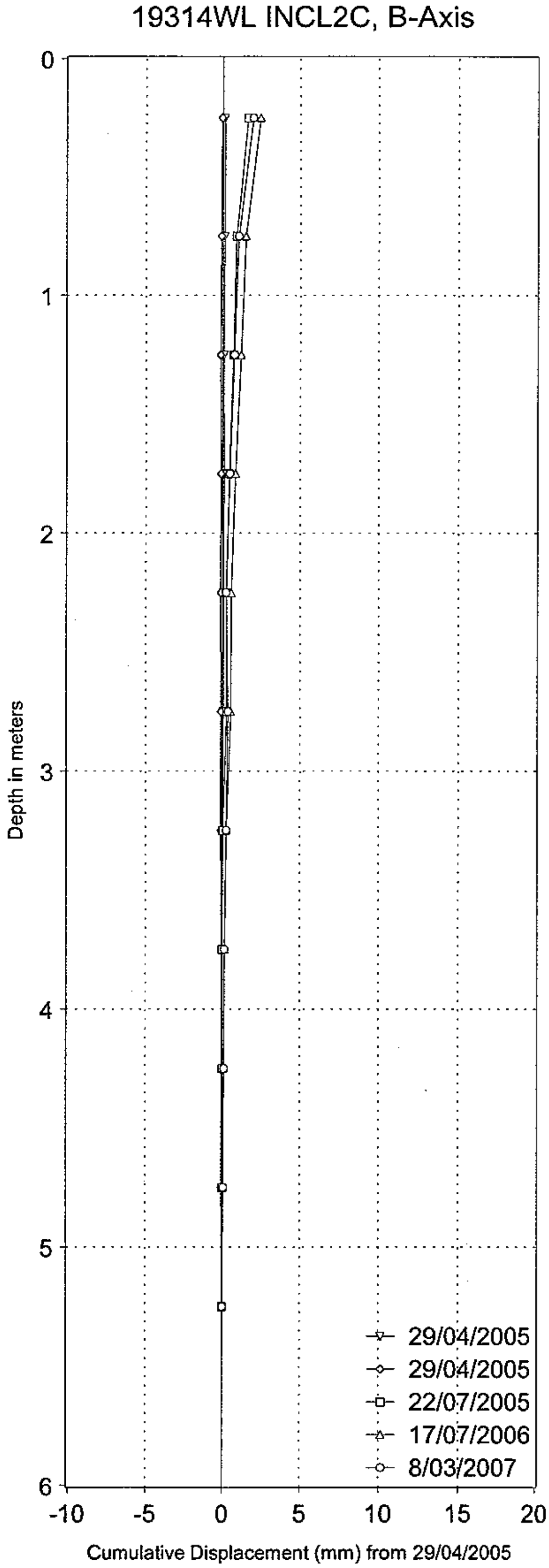


Jeffery and Katauskas Pty Ltd
CLIENT: Water Research Laboratory UNSW
PROJECT: Inclinometer Monitoring
LOCATION: Pilot Hill, Yamba, NSW

Job No: 19314WL
BH/INCL No: INCL1C

Figure No: 5

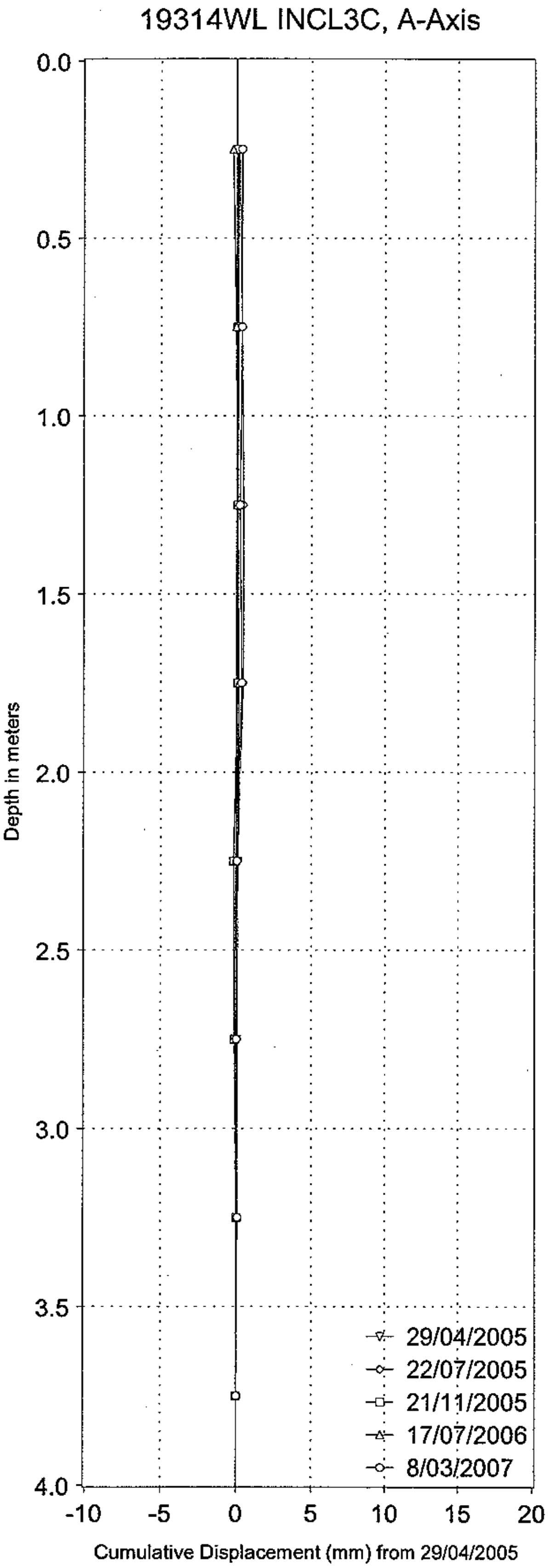


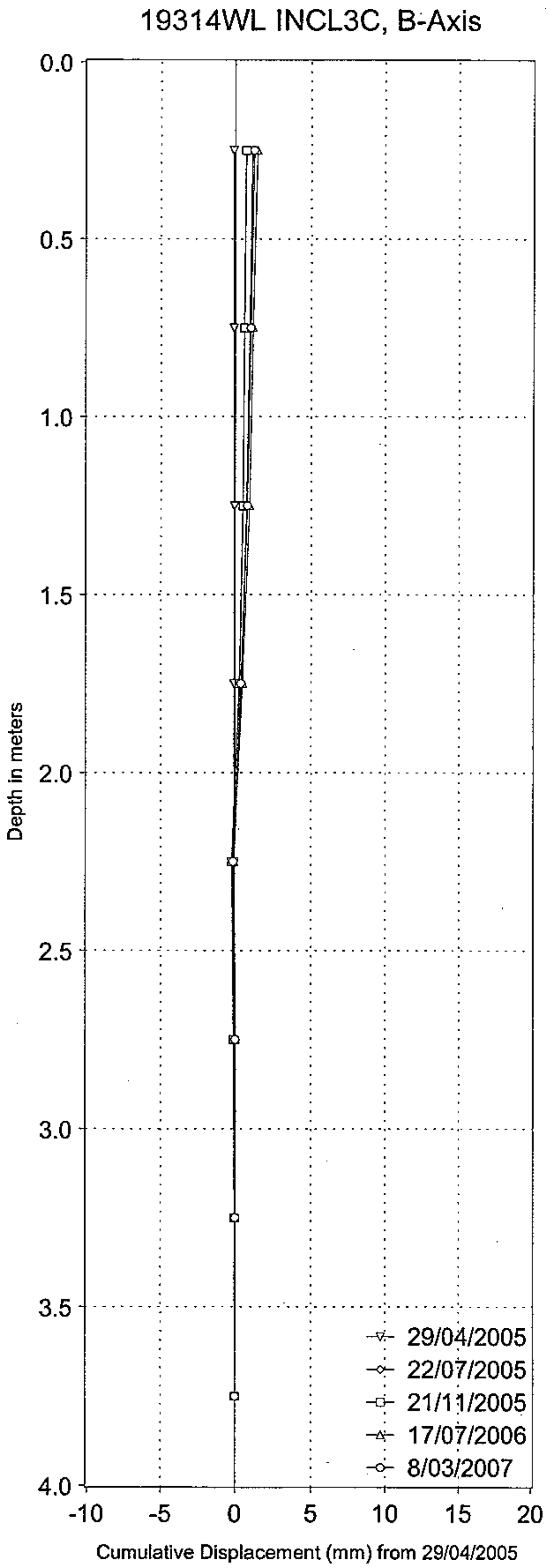


Jeffery and Katauskas Pty Ltd
CLIENT: Water Research Laboratory UNSW
PROJECT: Inclinometer Monitoring
LOCATION: Pilot Hill, Yamba, NSW

Job No: 19314WL
BH/INCL No: INCL2C

Figure No: 6





Jeffery and Katauskas Pty Ltd
CLIENT: Water Research Laboratory UNSW
PROJECT: Inclinometer Monitoring
LOCATION: Pilot Hill, Yamba, NSW

Job No: 19314WL
BH/INCL No: INCL3C

Figure No: 7

APPENDIX C

GROUNDWATER LOGGER HYDROGRAPHS

Hydrographs

Groundwater Stations
Yamba Hill

May 2007

Mathew Baker
Groundwater Data Collection Services Pty Ltd
ABN:16 083 771 242
PO Box 371
Casino NSW 2470

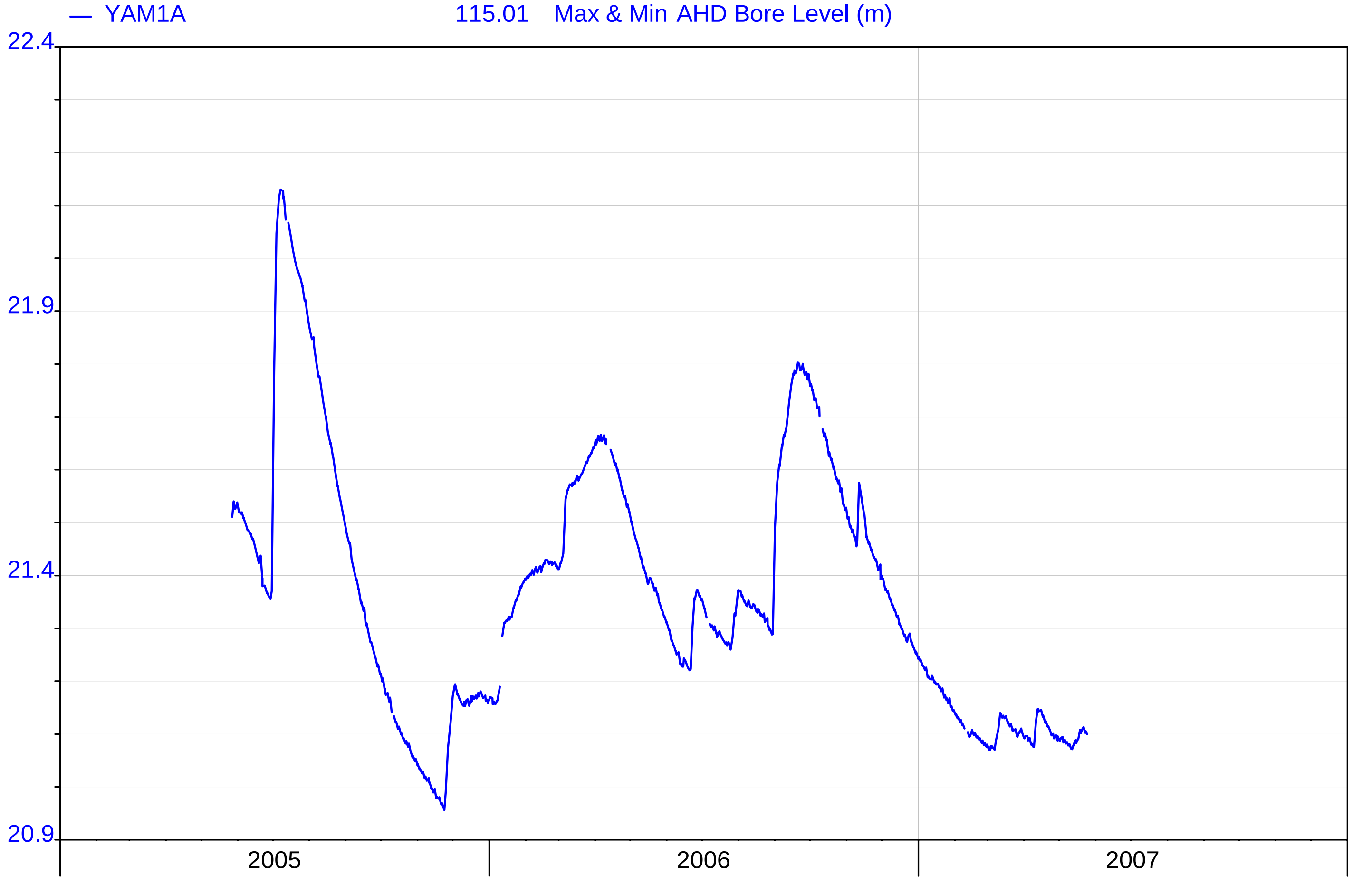
Phone: 02 66 675164
Fax: 02 66 675184
Mobile: 0418 104 234
Email: matgdcs@nor.com.au

Groundwater Data Collection Services

HYPLOT V125 Output 30/05/2007

Period 3 Year   Plot Start 00:00_01/01/2005   2005-08

Interval 2 Day   Plot End 00:00_01/01/2008

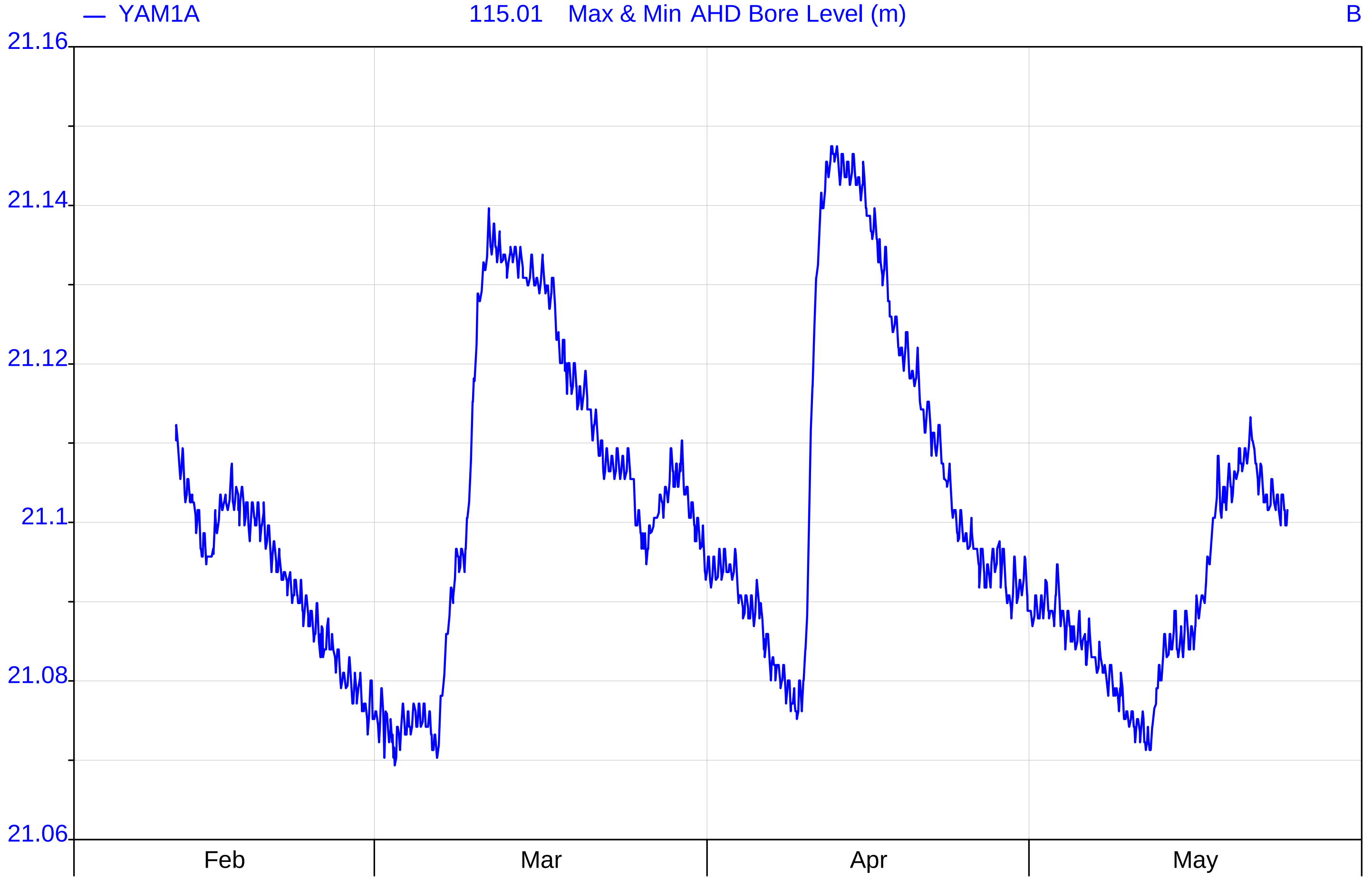


Groundwater Data Collection Services

HYPLOT V125 Output 30/05/2007

Period 4 Month Plot Start 00:00_01/02/2007 2007

Interval 4 Hour Plot End 00:00_01/06/2007



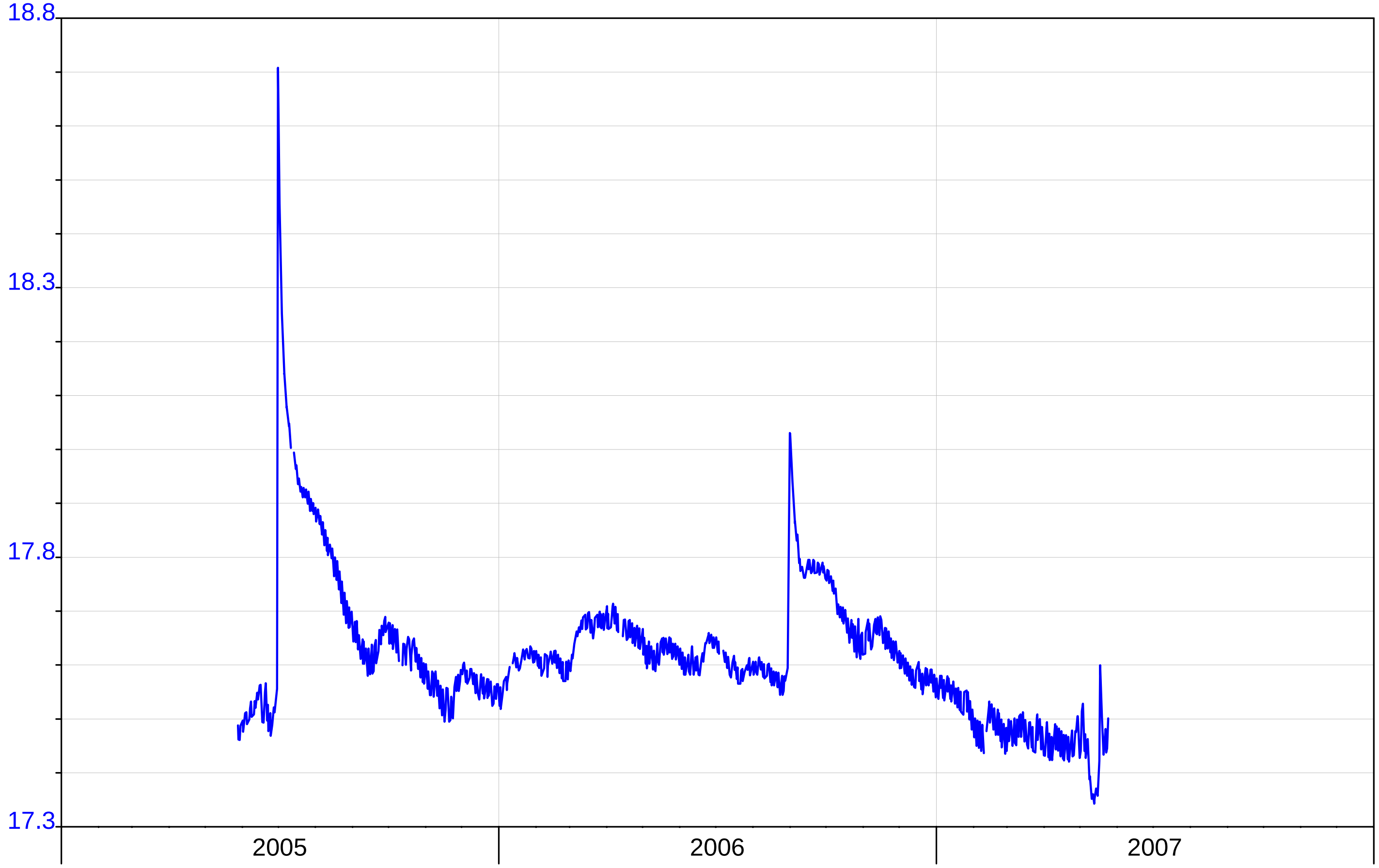
Groundwater Data Collection Services

HYPLOT V125 Output 30/05/2007

Period 3 Year Plot Start 00:00_01/01/2005 2005-08

Interval 2 Day Plot End 00:00_01/01/2008

— YAM1B 115.01 Max & Min AHD Bore Level (m)



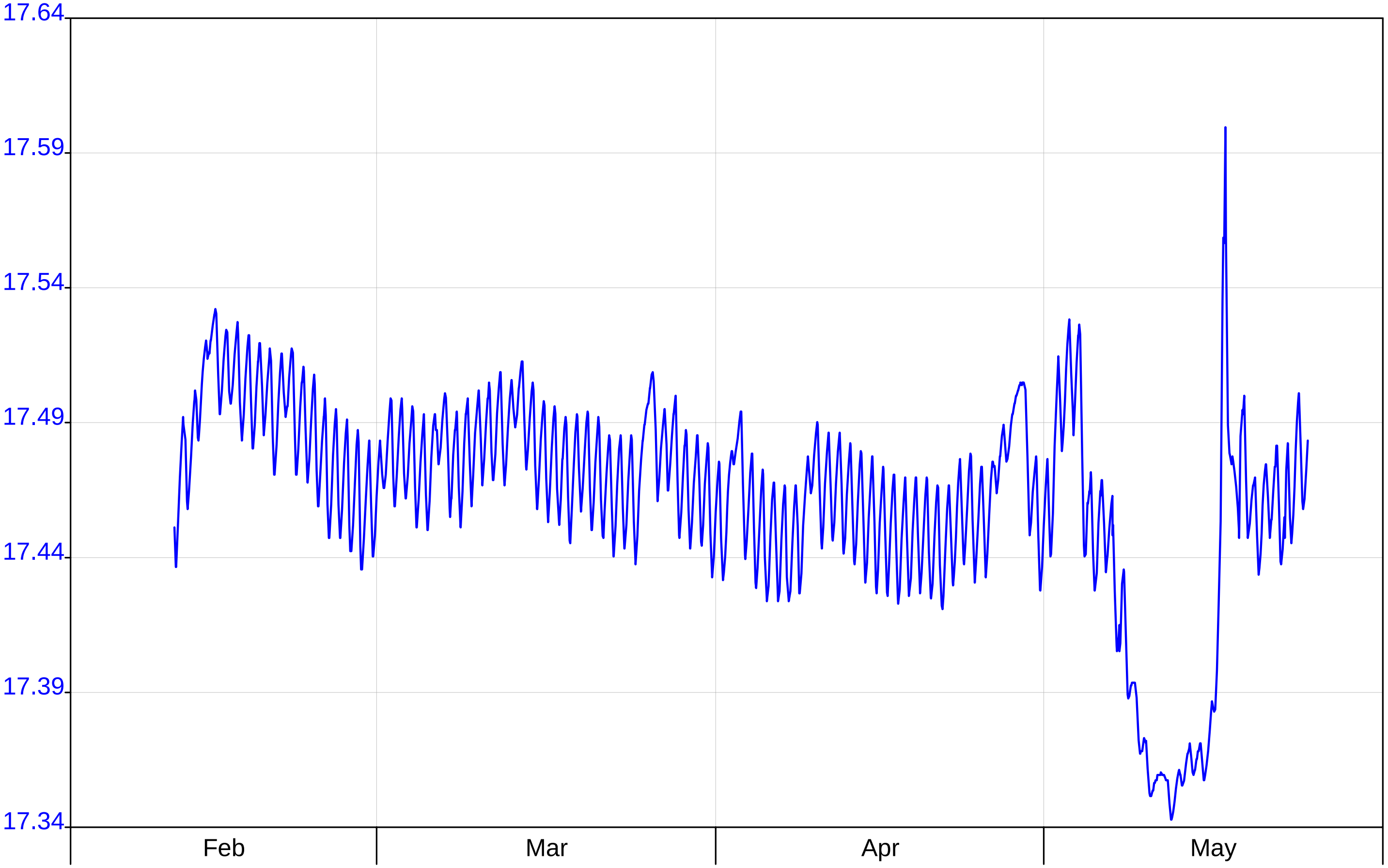
Groundwater Data Collection Services

HYPLOT V125 Output 30/05/2007

Period 4 Month Plot Start 00:00_01/02/2007 2007

Interval 4 Hour Plot End 00:00_01/06/2007

— YAM1B 115.01 Max & Min AHD Bore Level (m) B



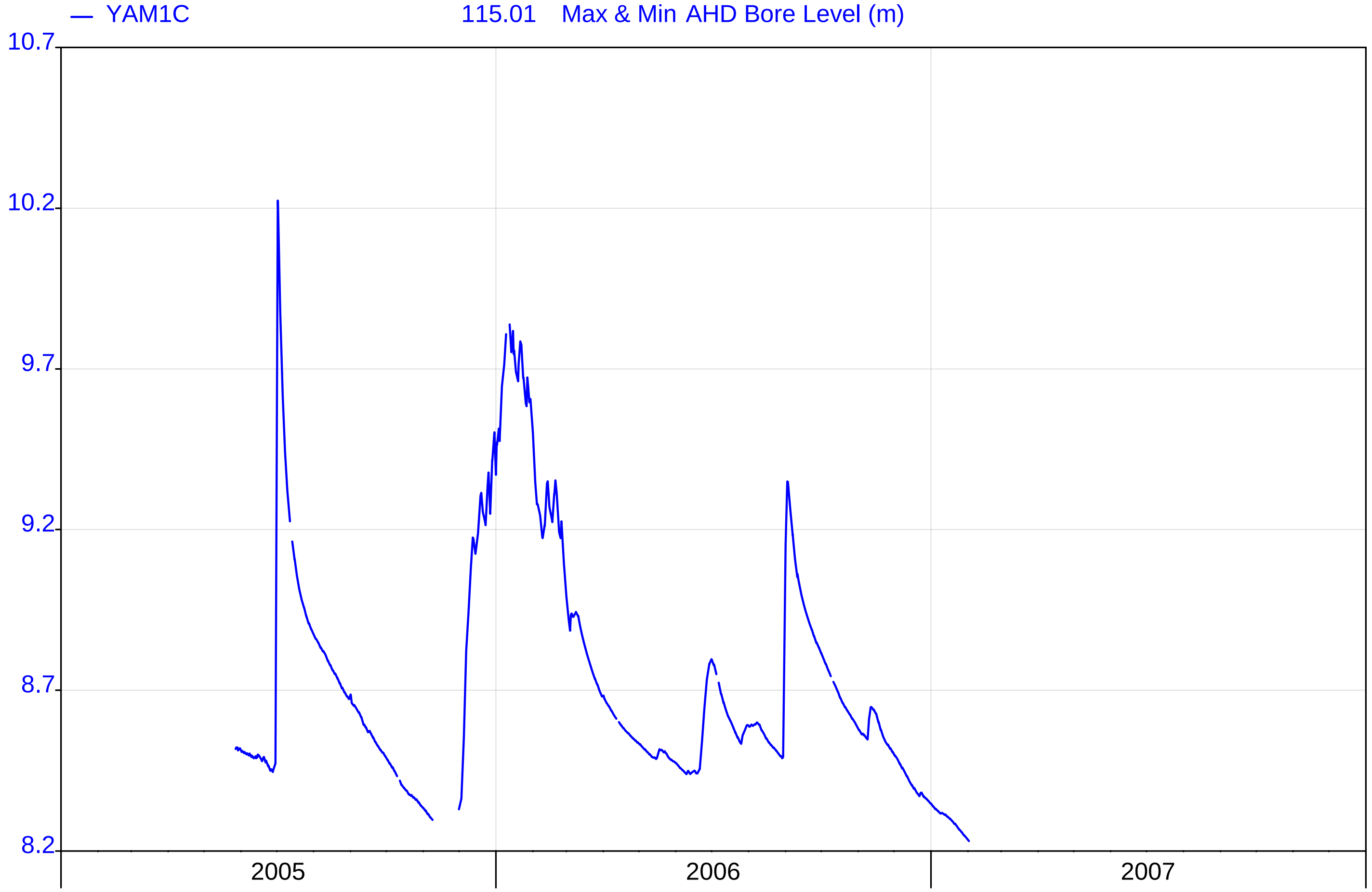
Groundwater Data Collection Services

HYPLOT V125 Output 30/05/2007

Period 3 Year Plot Start 00:00_01/01/2005

Interval 2 Day Plot End 00:00_01/01/2008

2005-08

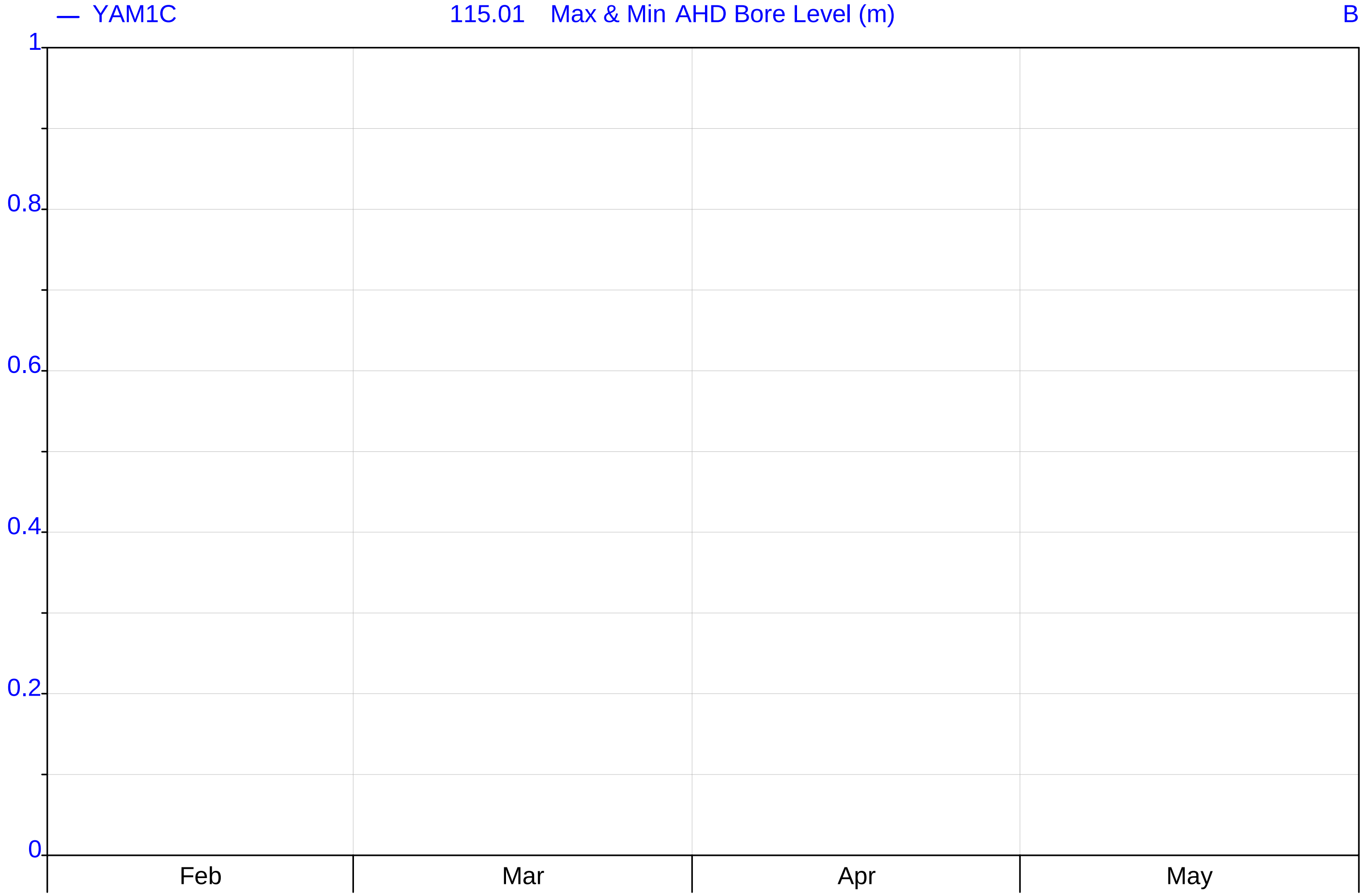


Groundwater Data Collection Services

HYPLOT V125 Output 30/05/2007

Period 4 Month Plot Start 00:00_01/02/2007 2007

Interval 4 Hour Plot End 00:00_01/06/2007



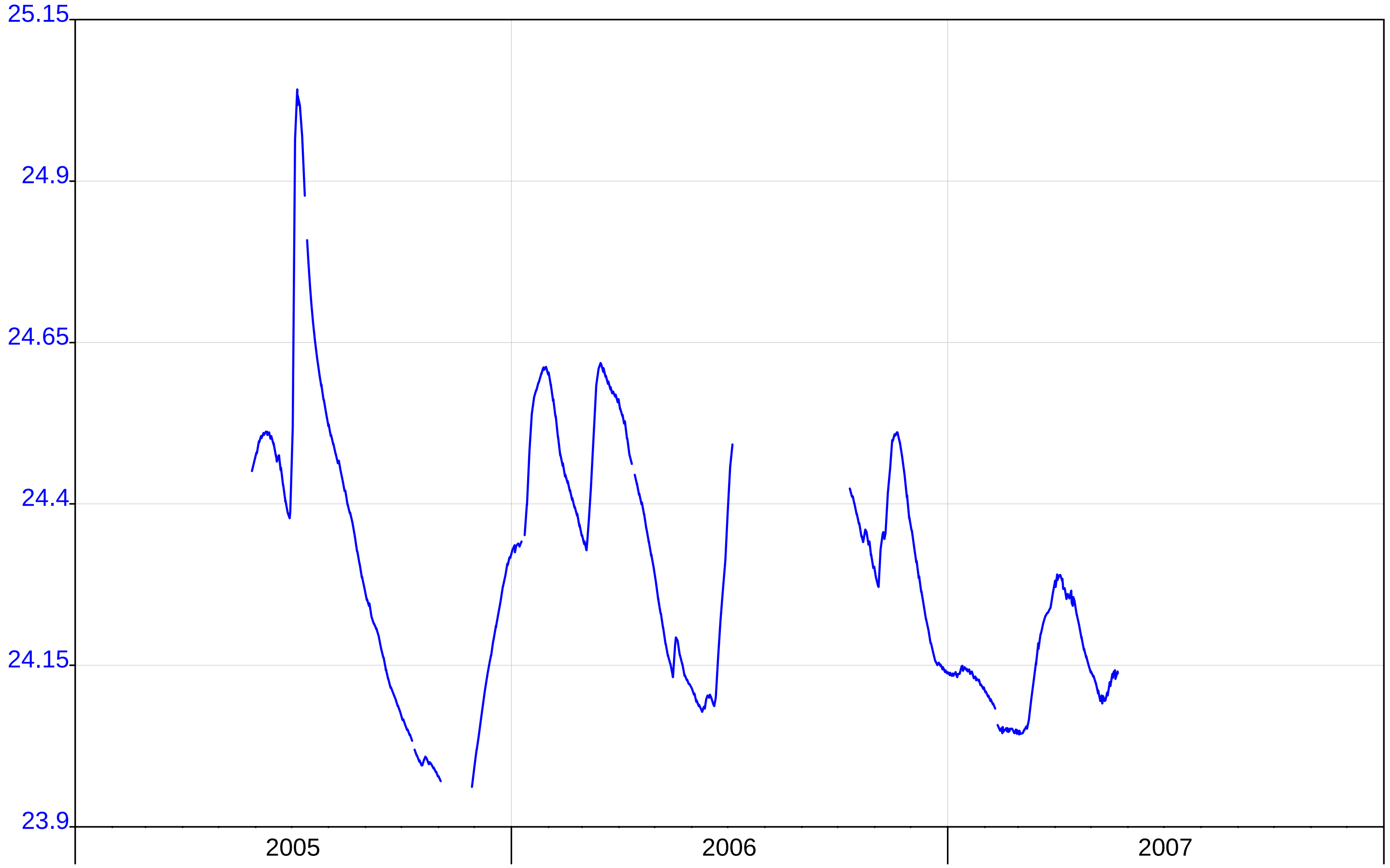
Groundwater Data Collection Services

HYPLOT V125 Output 30/05/2007

Period 3 Year   Plot Start 00:00_01/01/2005   2005-08

Interval 2 Day   Plot End 00:00_01/01/2008

— YAM2A   115.01   Max & Min AHD Bore Level (m)



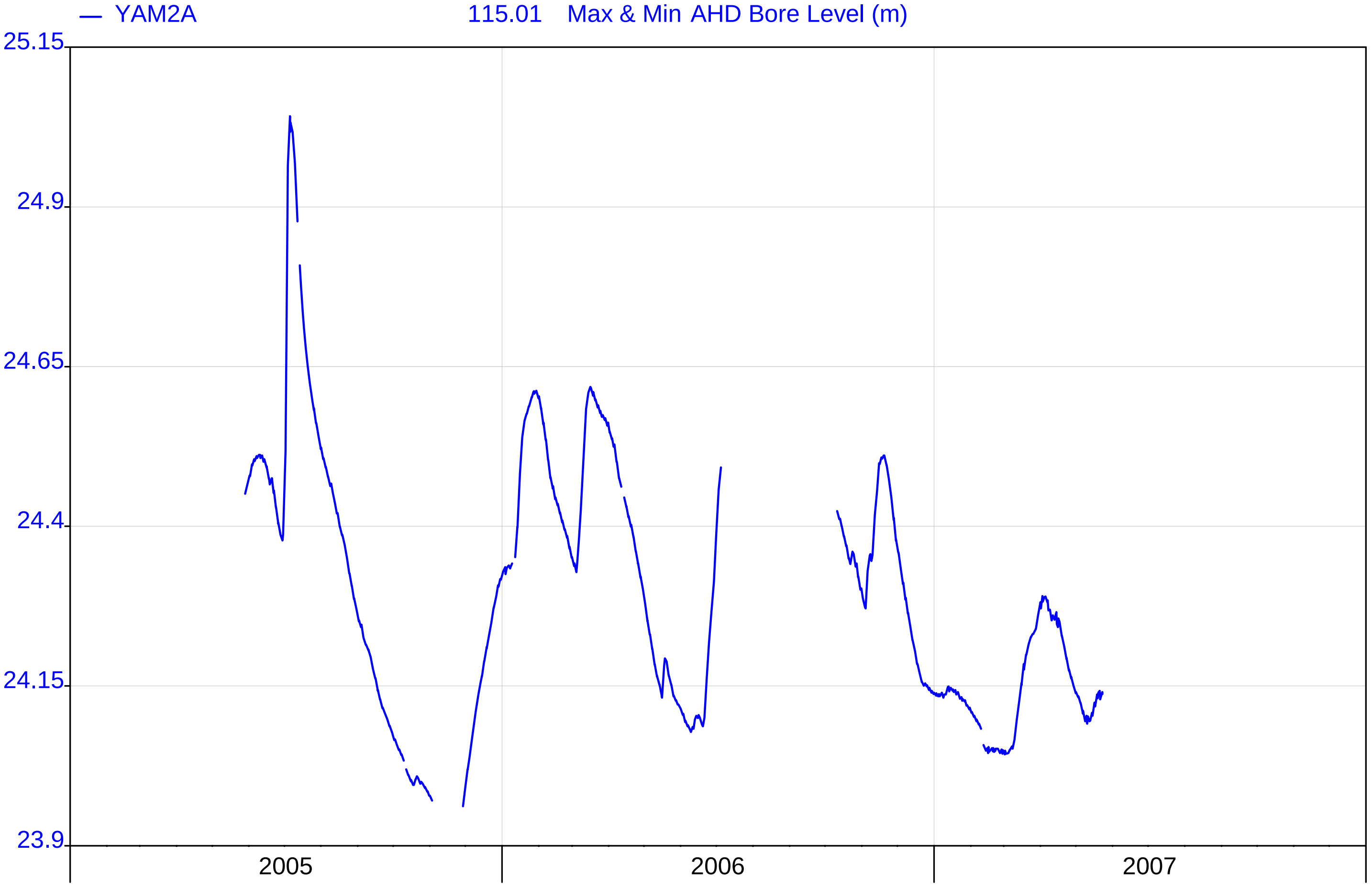
Groundwater Data Collection Services

HYPLOT V125 Output 30/05/2007

Period 3 Year Plot Start 00:00_01/01/2005

2005-08

Interval 2 Day Plot End 00:00_01/01/2008

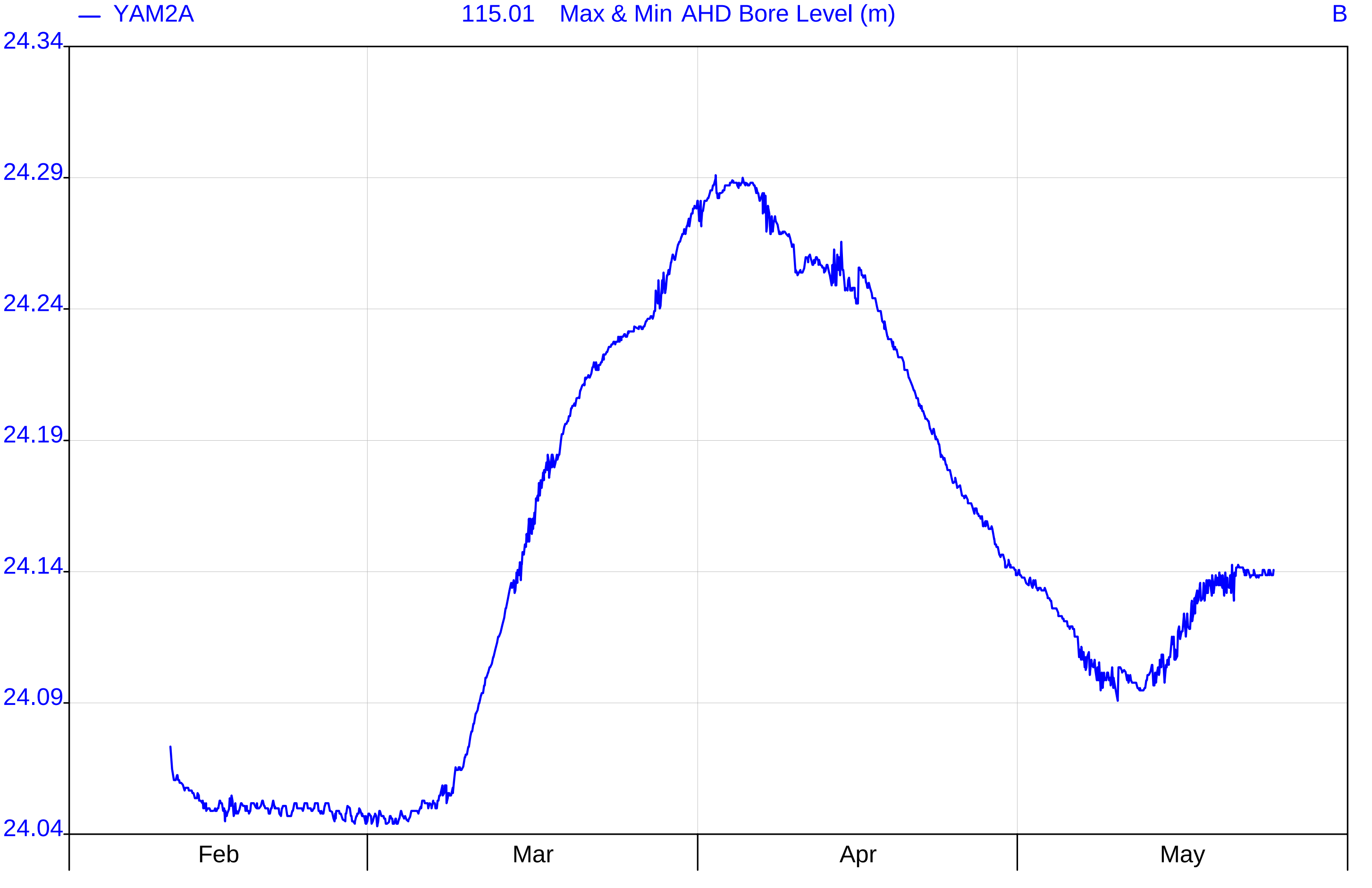


Groundwater Data Collection Services

HYPLOT V125 Output 30/05/2007

Period 4 Month Plot Start 00:00_01/02/2007 2007

Interval 4 Hour Plot End 00:00_01/06/2007

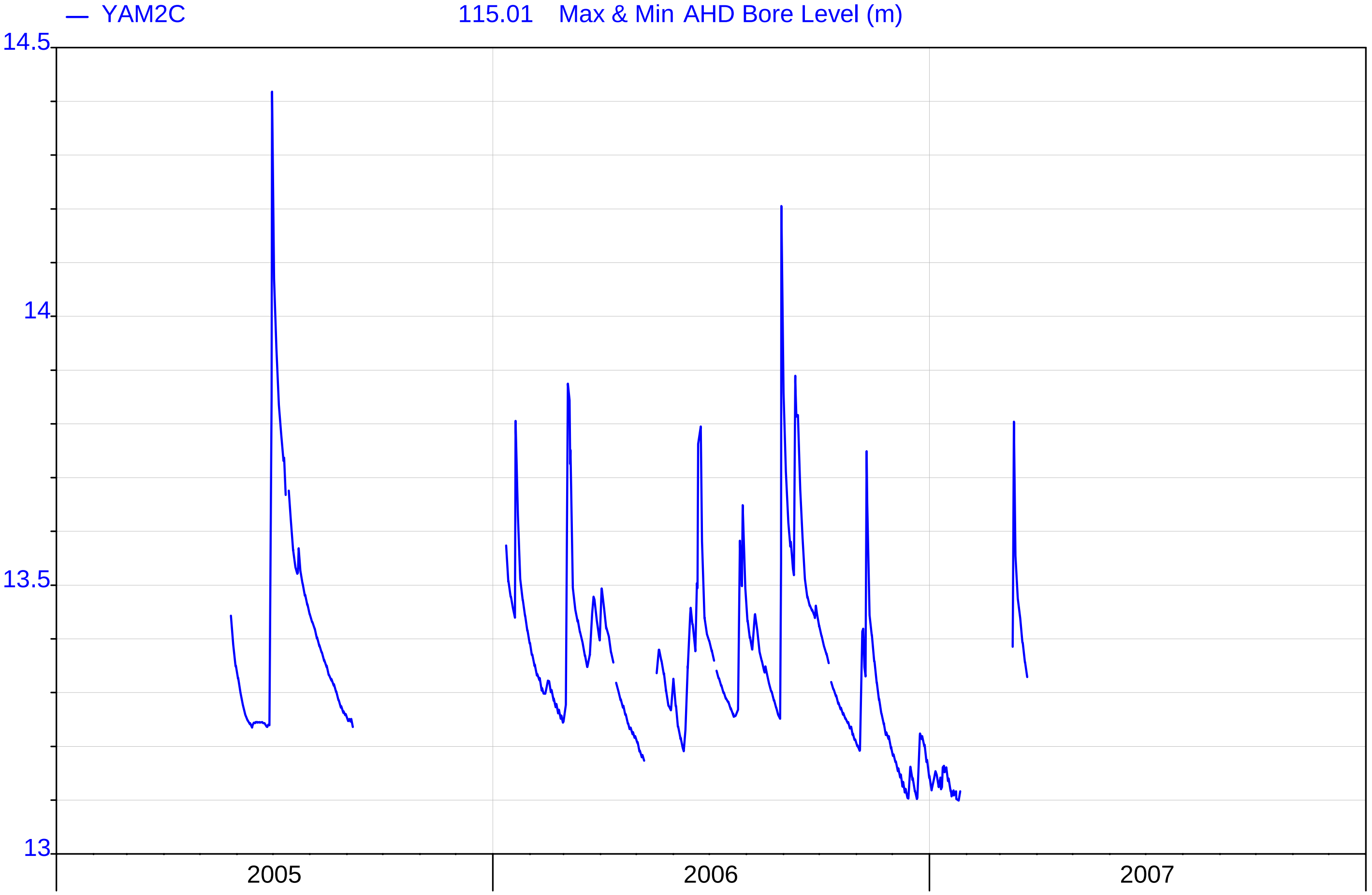


Groundwater Data Collection Services

HYPLOT V125 Output 30/05/2007

Period 3 Year Plot Start 00:00_01/01/2005 2005-08

Interval 2 Day Plot End 00:00_01/01/2008



Groundwater Data Collection Services

HYPLOT V125 Output 30/05/2007

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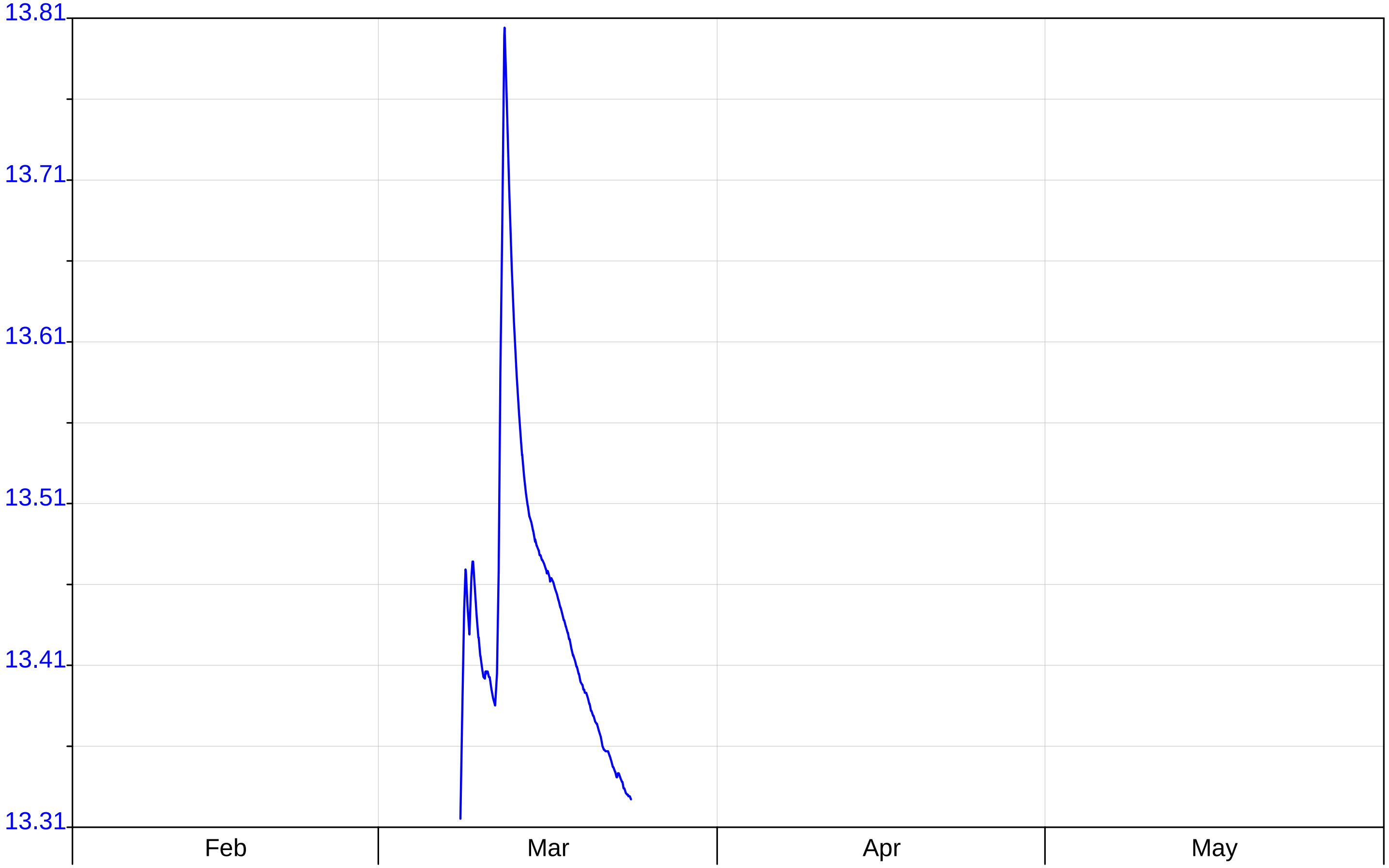
Interval 4 Hour Plot End 00:00_01/06/2007

— YAM2C 115.01 Max & Min AHD Bore Level (m) B



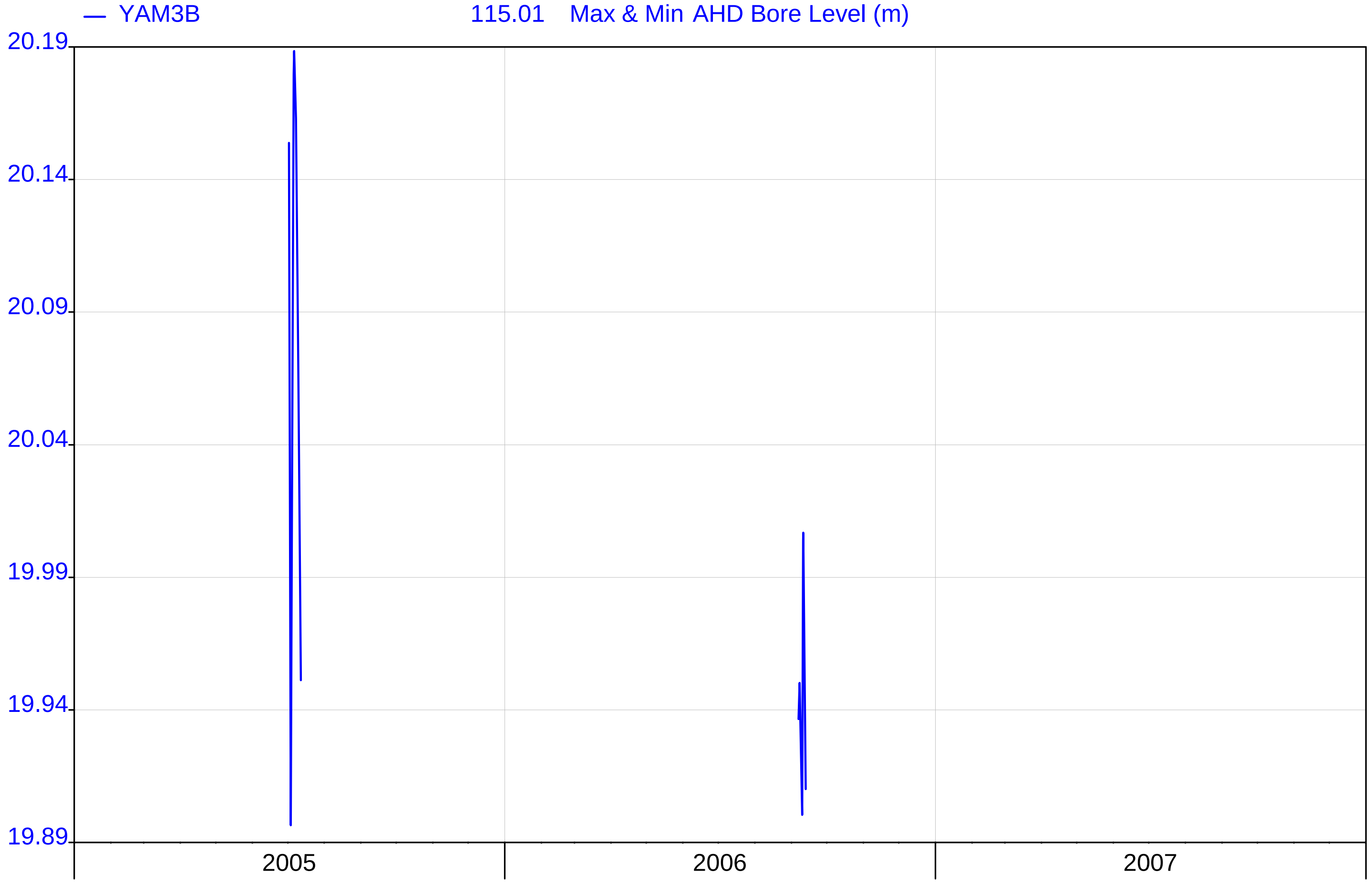
Groundwater Data Collection Services

HYPLOT V125 Output 30/05/2007

Period 3 Year Plot Start 00:00_01/01/2005

Interval 2 Day Plot End 00:00_01/01/2008

2005-08



Groundwater Data Collection Services

HYPLOT V125 Output 30/05/2007

Period 4 Month Plot Start 00:00_01/02/2007 2007

Interval 4 Hour Plot End 00:00_01/06/2007

— YAM3B 115.01 Max & Min AHD Bore Level (m) B

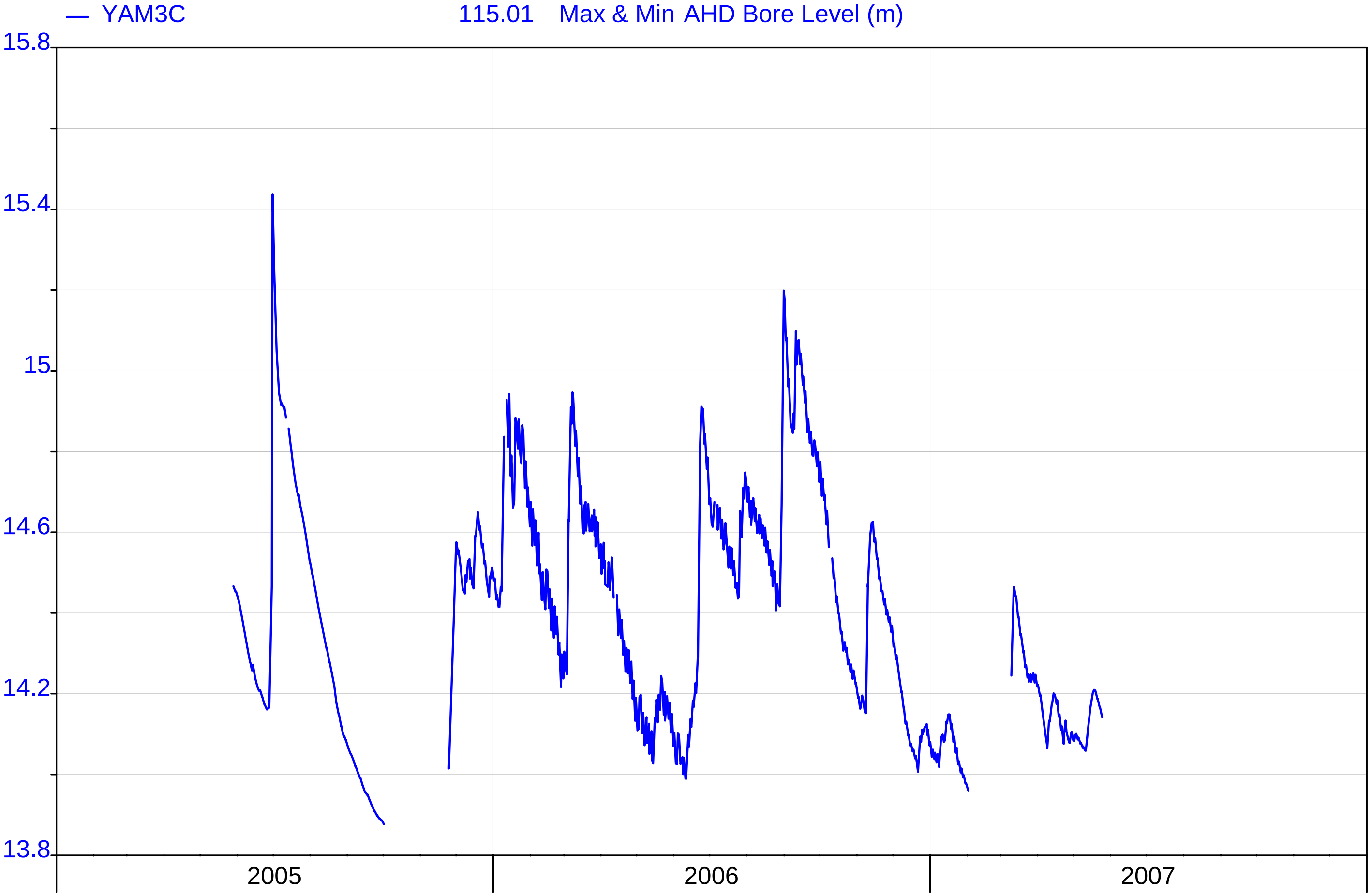
Groundwater Data Collection Services

HYPLOT V125 Output 30/05/2007

Period 3 Year Plot Start 00:00_01/01/2005

Interval 2 Day Plot End 00:00_01/01/2008

2005-08



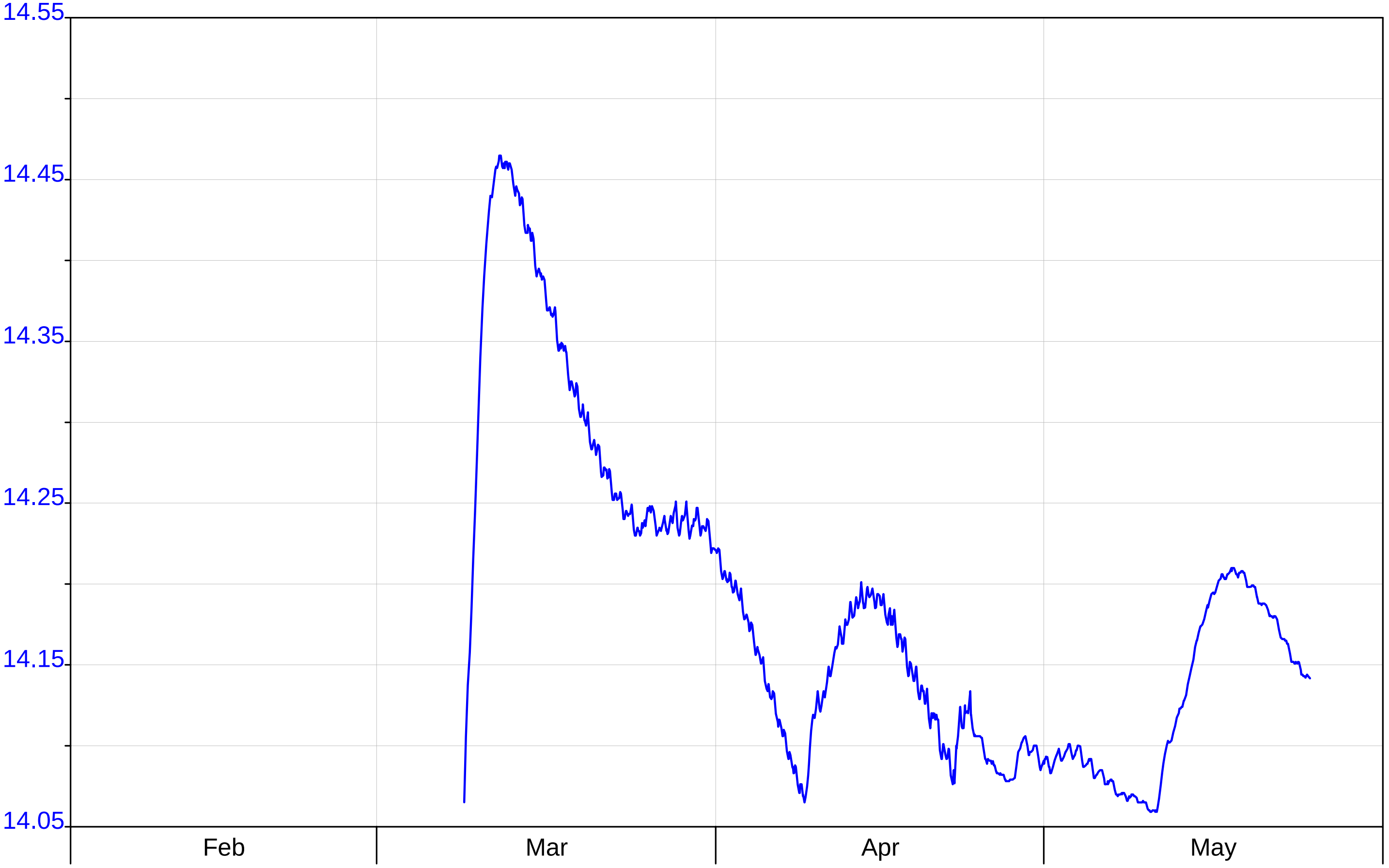
Groundwater Data Collection Services

HYPLOT V125 Output 30/05/2007

Period 4 Month Plot Start 00:00_01/02/2007 2007

Interval 4 Hour Plot End 00:00_01/06/2007

— YAM3C 115.01 Max & Min AHD Bore Level (m) B



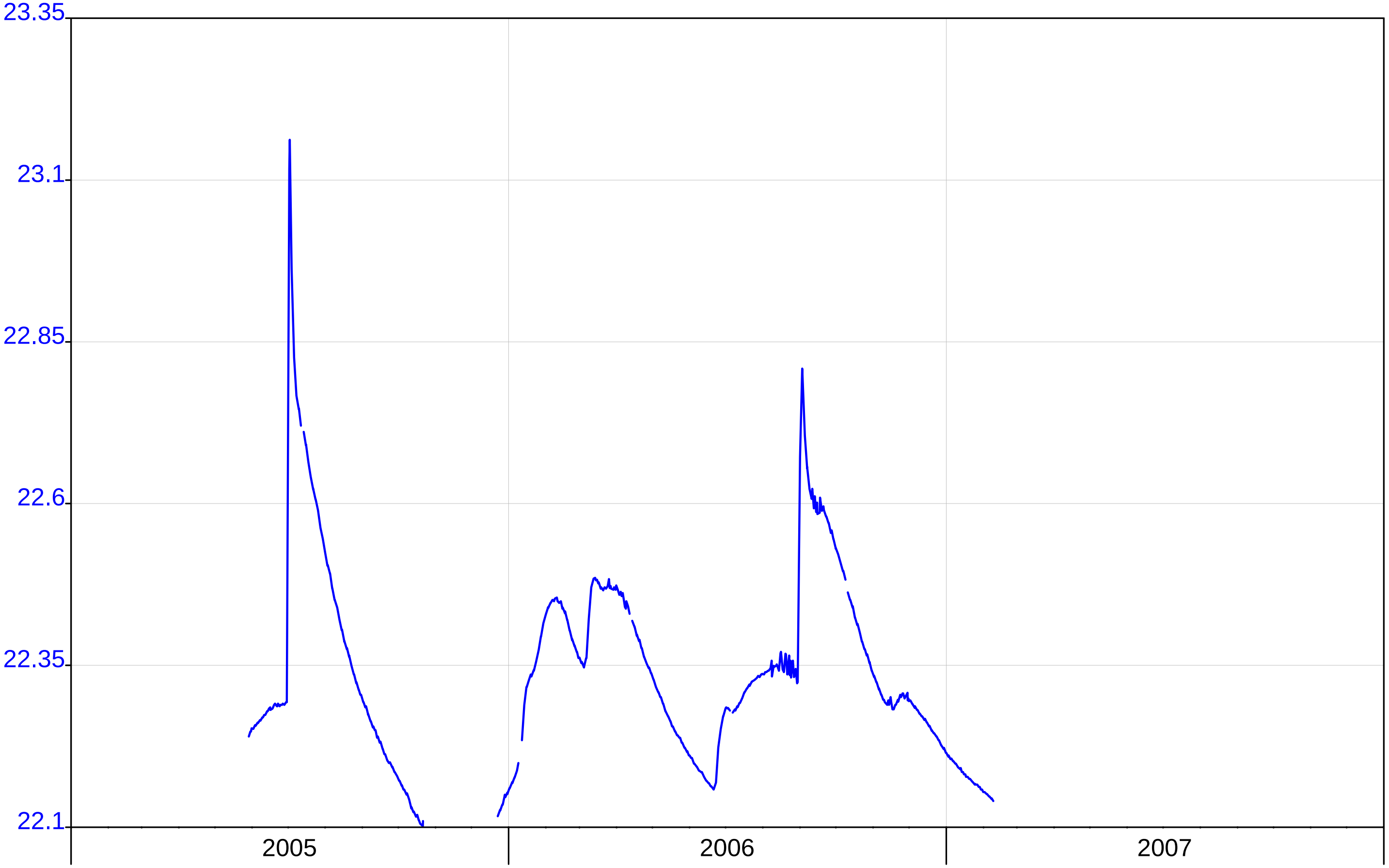
Groundwater Data Collection Services

HYPLOT V125 Output 30/05/2007

Period 3 Year Plot Start 00:00_01/01/2005 2005-08

Interval 2 Day Plot End 00:00_01/01/2008

— YAMMSA1 115.01 Max & Min AHD Bore Level (m)



GROUND Data Collection Services WATER

Groundwater Data Collection Services

HYPLOT V125 Output 30/05/2007

Period 4 Month Plot Start 00:00_01/02/2007 2007

Interval 4 Hour Plot End 00:00_01/06/2007

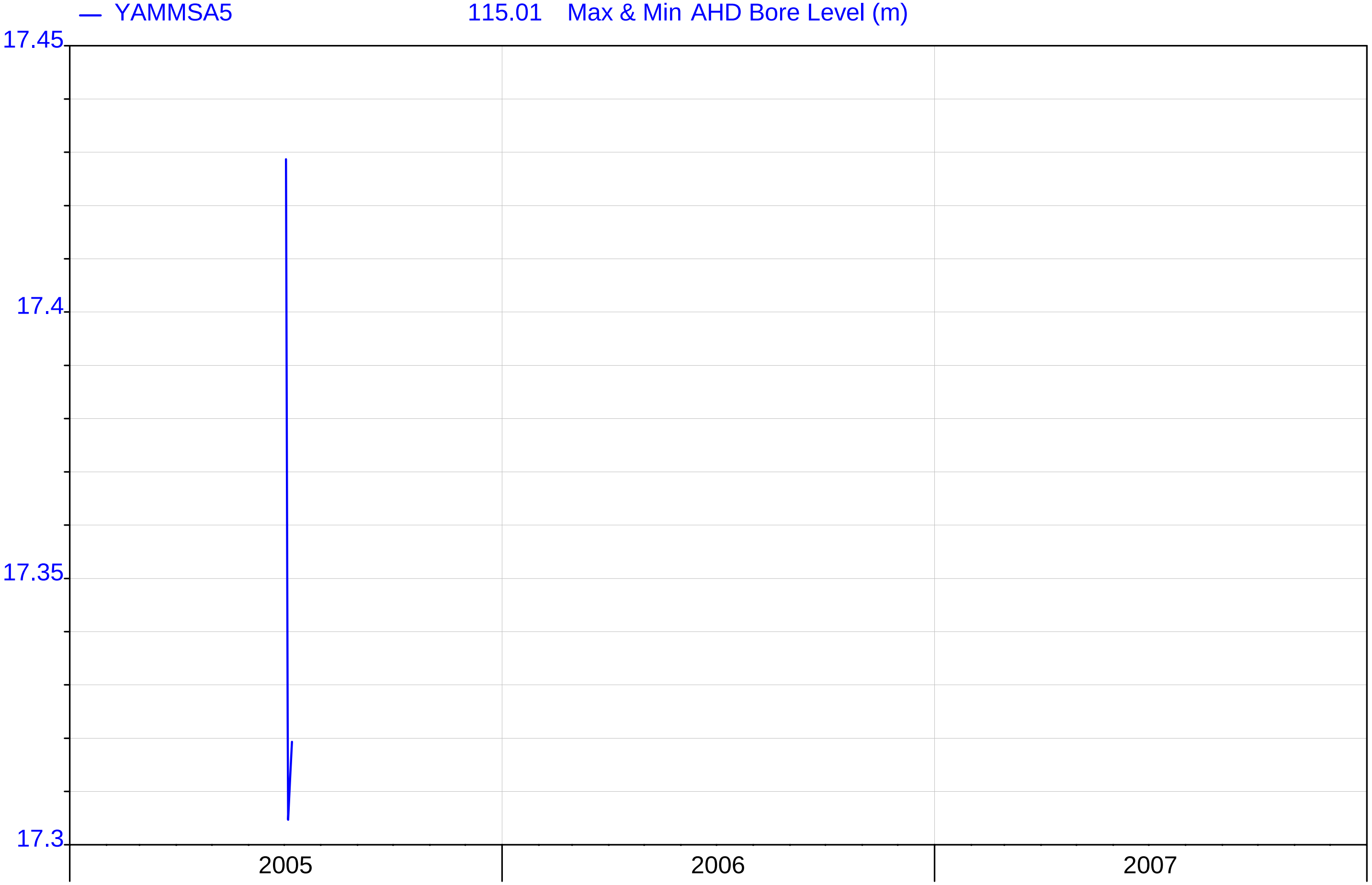
— YAMMSA1 115.01 Max & Min AHD Bore Level (m) B

Groundwater Data Collection Services

HYPLOT V125 Output 30/05/2007

Period 3 Year Plot Start 00:00_01/01/2005 2005-08

Interval 2 Day Plot End 00:00_01/01/2008



Groundwater Data Collection Services

HYPLOT V125 Output 30/05/2007

Period 4 Month Plot Start 00:00_01/02/2007 2007

Interval 4 Hour Plot End 00:00_01/06/2007

— YAMMSA5 115.01 Max & Min AHD Bore Level (m) B

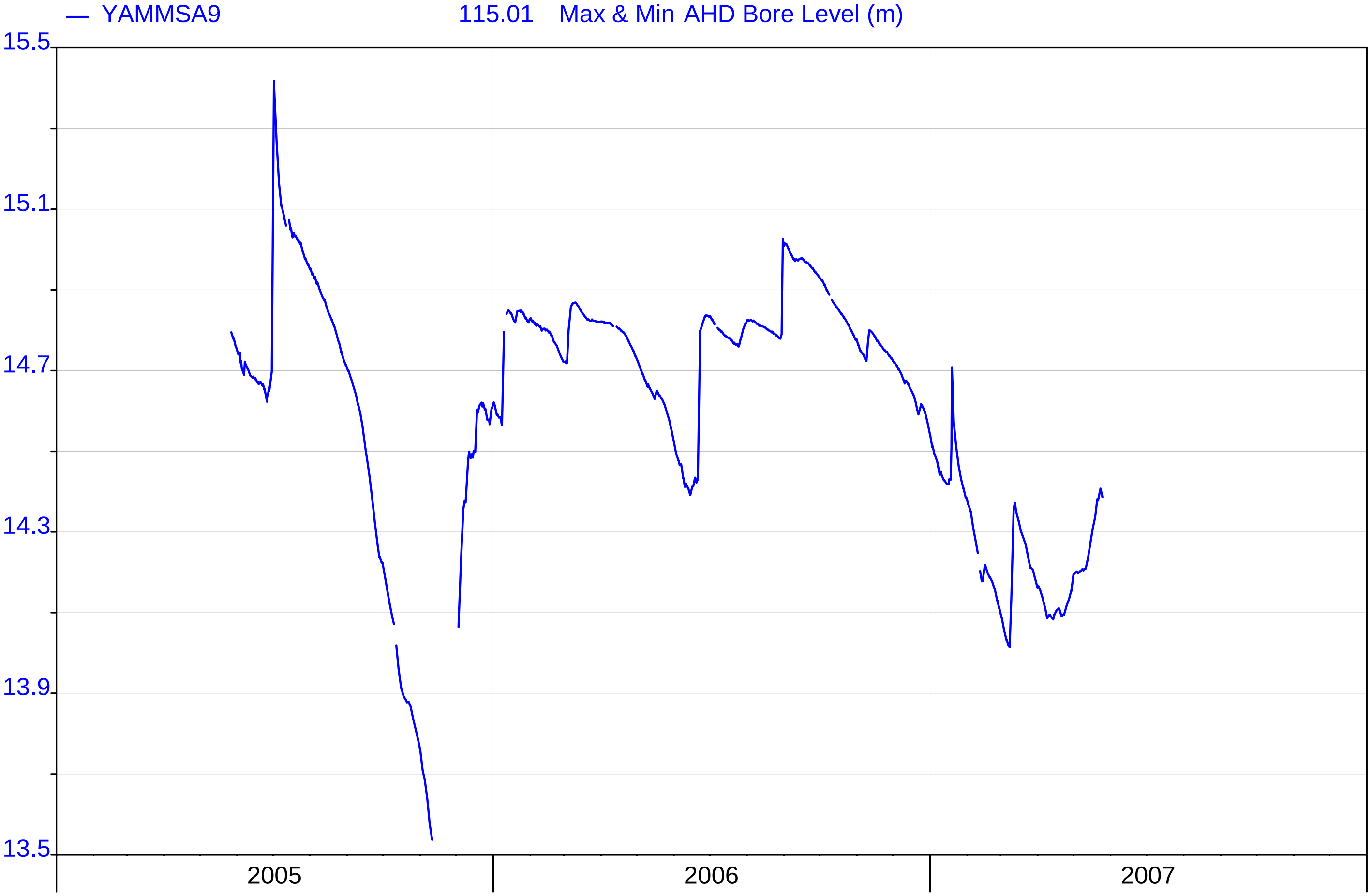
Groundwater Data Collection Services

HYPLOT V125 Output 30/05/2007

Period 3 Year Plot Start 00:00_01/01/2005

2005-08

Interval 2 Day Plot End 00:00_01/01/2008

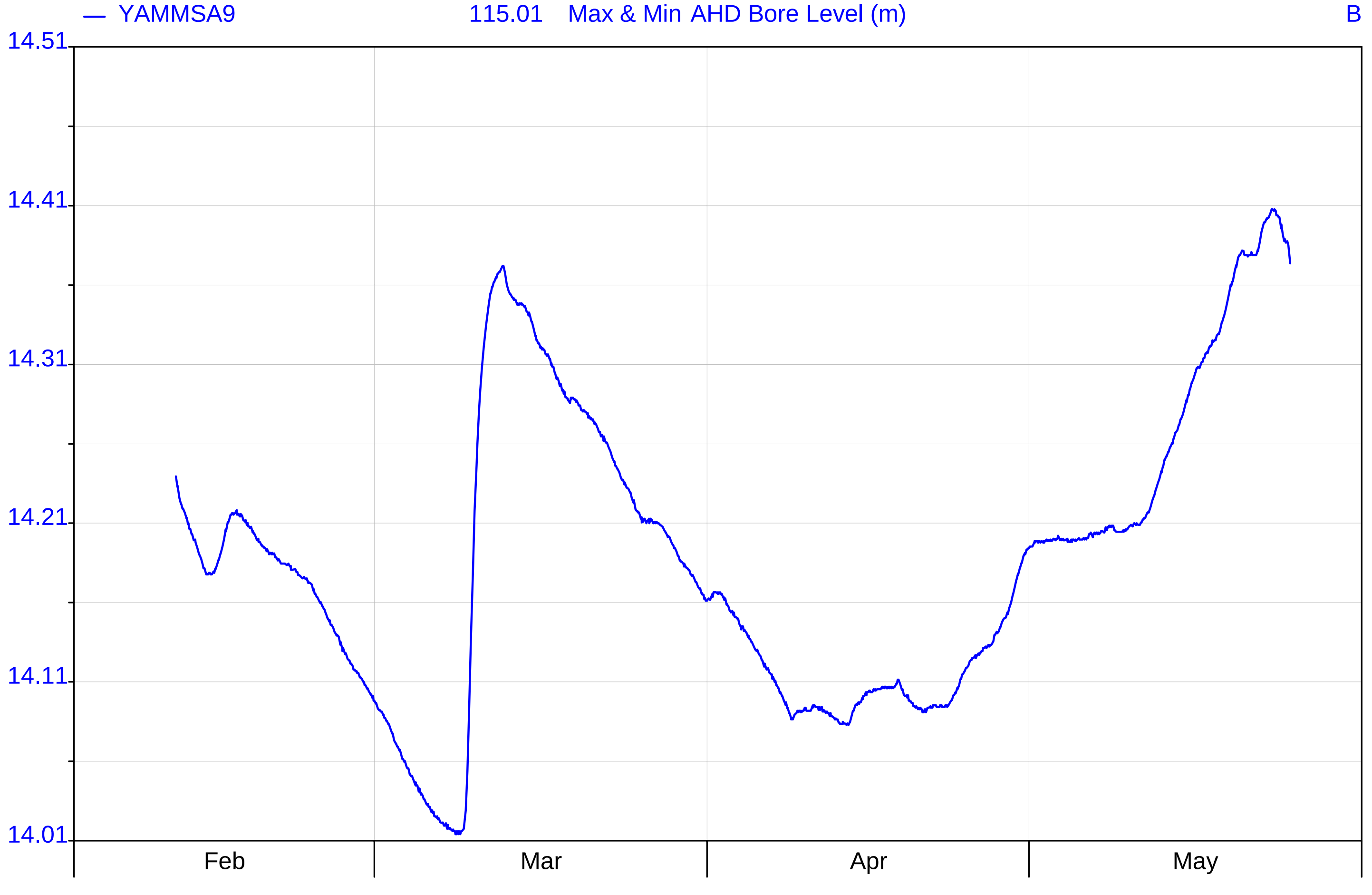


Groundwater Data Collection Services

HYPLOT V125 Output 30/05/2007

Period 4 Month Plot Start 00:00_01/02/2007 2007

Interval 4 Hour Plot End 00:00_01/06/2007



APPENDIX D

NEWSLETTER NO. 4

YAMBA HILL

Groundwater Monitoring and Investigation

The UNSW Water Research Laboratory is pleased to provide a progress update on this project to stakeholders including local residents. This is the latest in a series of newsletters that are available on the project website as follows: Newsletter No. 1 (March, 2005) introduced the purpose and background to the study, the scope of work and team members. Newsletter No. 2 (May, 2005) reported on drilling and groundwater monitoring installations as part of the landslide risk management process. Newsletter No. 3 (August, 2005) discussed the response measured to a 1 in 80 year rain event in June/July 2005.

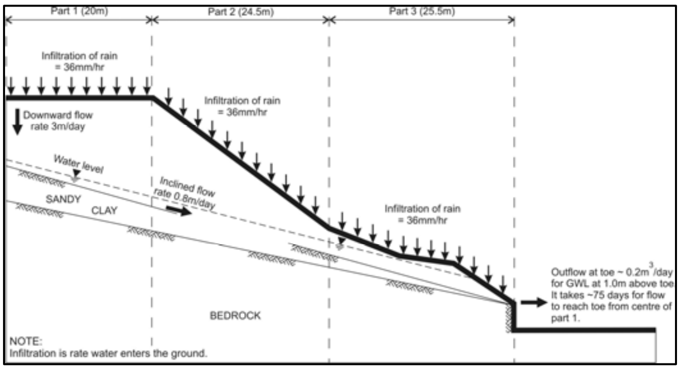
NEWS

Project reports for the monitoring period May 2005 to 2007 have been presented to council, and a community meeting to discuss the project findings was scheduled for 6th November. A summary report (see attached) was prepared for the community and the NSW Coastal Conference to be held in Yamba (7-9th Nov).

KEY FINDINGS

No landslides were recorded during the monitoring period between May 2005 and May 2007, although inclinometer monitoring showed slow creep near the toe of the slope (Site 2C). High groundwater levels are the most common trigger for landslides. A review of groundwater level results for a 2 year period have shown the most rapid groundwater level response to a rainfall event occurred at mid-slope (~10 to 12 hours), followed by the toe of slope (17 to 34 hours). There was a considerable lag before groundwater levels responded to rainfall at the top of the slope, and for drainage of groundwater from the slope to reduce the risk of landslides. These results support the initial conceptual model describing how water moves through the hill (see Figure below).

Jeffery and Katauskas Pty Ltd have used the monitoring data to assess hillslope stability. The designated landslide risk zones (LRZs) have been confirmed by this work. The calculated probability of earthslides and scour at the toe of the slope was in the range of 1 in 20 years to 1 in 250 years. For the steeper hillslide slopes the probability was higher at 1 in 20 years to 1 in 200 years. The risk of rapid landslides within Zone 1a and 1b is considered to be unacceptable.



RECOMMENDATIONS

Various slope treatment and stabilisation options should be investigated in more detail for implementation as a matter of priority. In the meantime, monitoring of groundwater levels and inclinometers should continue. WRL and Jeffery and Katauskas Pty Ltd have recommended that council should continue with the emergency levels and management strategies until more permanent stablisation measures are adopted.

FURTHER DETAILS

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LIVING ON THE EDGE – INVESTIGATING LANDSLIDE RISK IN A COASTAL COMMUNITY

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Introduction and background

The coastline is constantly being reshaped by the forces of nature. As it is also a desirable zone for habitation, an important component of any coastline management plan includes assessing landslide risk and implementation of appropriate management strategies.

A project to review landslide risk on the slope above Main Beach, Yamba Hill was initiated by Clarence Valley Council with funding from the Natural Disaster Mitigation Program. This paper provides preliminary findings of the project including verification of a conceptual model of subsurface processes and groundwater levels that may trigger slope instability.

The project was designed specifically to include community consultation (eg. meetings, newsletters and a website) in parallel with technical investigations. Meetings with the community and individual residents were held prior to investigations. Information flow is actively maintained through newsletters and a website, with opportunities for community feedback.

Yamba Hill is a coastal dune approximately 30 m high that overlies weathered sandstone and a cliff approximately 6-8 m high above Main Beach. The slopes above the cliff line are 18-35° and vegetated by scrub and bushes. Residential lots and the Pacific Hotel are located on a flatter bench area which rises from Marine Parade behind the Surf Life Saving Club.

Landslide risk zones (LRZs) were ranked as a function of consequence multiplied by probability. Higher landslide risk is associated with developed coastal areas due to the consequences of potential landslips to people and property. This current study builds upon an earlier assessment for the Yamba Coastline Management Study (MHL, 2003) which included a geotechnical assessment (J&K, 2000). Evidence of slope instability in this area is evident during site walkovers, including fractured pathways and downwards creep of retaining walls. The site has a history of slope instability, with known landslides used to derive the probability of instability.

The Water Research Laboratory (WRL) team worked in association with Jeffery and Katauskas (J&K) Pty Ltd and Groundwater Data Collection Services (GDCS) on the Yamba Hill project.

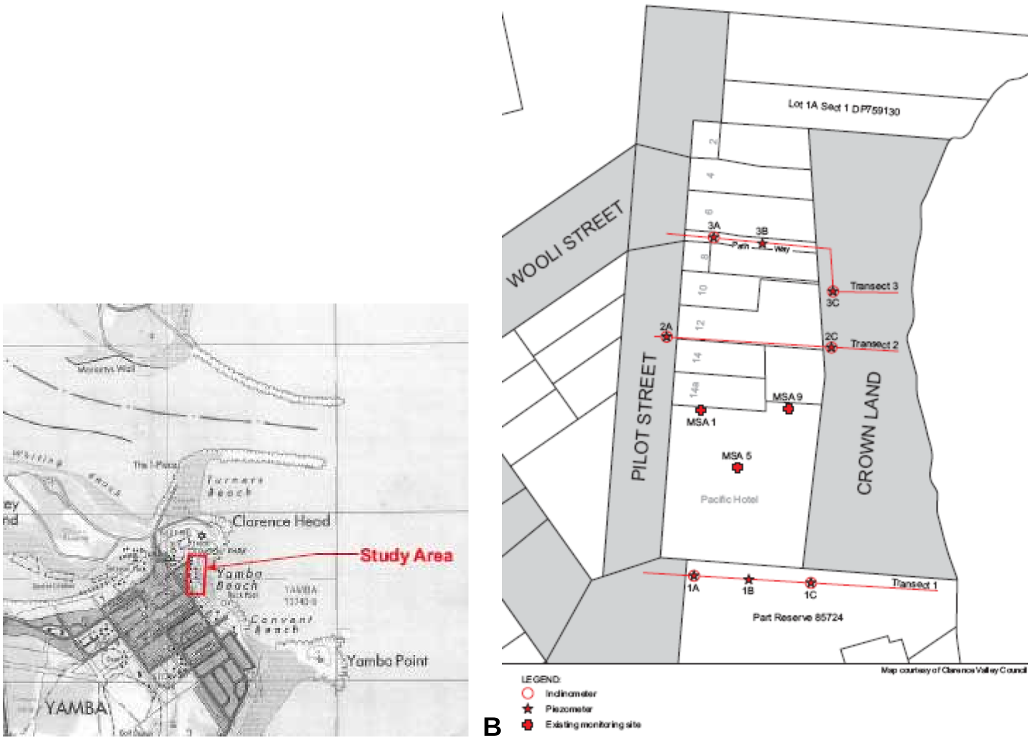

Fig 1 Location map of A) Yamba Hill study area B) Monitoring transects, piezometers and inclinometer installations.

Fig 2 View of the study area on Yamba Hill pictured from Yamba Beach

Community consultation

Community involvement in the project has been invited since the inception using a range of communications. This critical aspect of the project is continuing, with the current emphasis on sharing the findings of the slope stability assessment completed after 2 years of monitoring and seeking feedback from residents and stakeholders. To date, the project has included two community meetings held at the Yamba Surf Club, development of a website (www.wrl.unsw.edu.au/yamba), three information newsletters, regular letter updates from CVC to residents and stakeholders, and a phone contact number for information.

Site investigations and monitoring methods

Site investigations were designed to target high risk areas identified during previous studies. Small landslips have historically occurred in dune sands underlain by weathered clay and sandstone above Main Beach. Three monitoring transects were established on a high dune slope perpendicular to the beach (8 monitoring sites). Monitoring sites were located at the top of slope, mid-slope and at the toe of the sandy slope on a low sandstone cliff behind the beach (Table 1 and Figures 1 and 2).

Table 1
Summary of Monitoring Installations

Type	Site	Location		RL Surface	Depth
		mE	*mN*	*mAHD*	*mBGL*
Piezometer	1A	535239.34	6743756.73	29.47	11.58
	1B	535257.60	6743754.70	21.37	3.73
	1C	535279.74	6743753.97	11.96	3.95
	2A	535229.52	6743849.19	33.13	9.20
	2C	535284.18	6743843.10	14.62	1.39
	3A	535238.17	6743883.30	30.21	5.31
	3B	535264.08	6743882.30	23.87	3.80
	3C	535283.81	6743865.97	15.62	1.95
Inclinometer	1A	535240.19	6743756.10	29.41	19.05
	1C	535279.59	6743755.19	12.14	5.75
	2A	535230.52	6743850.19	33.18	13.85
	2C	535284.17	6743842.06	14.67	5.80
	3A	535239.28	6743883.50	30.24	11.30
	3C	535283.56	6743867.00	15.78	4.25

* All piezometers were 50 mm diameter PVC with machine slotted screened from 0.75 m above base.

Drilling, rock coring and testing of sediments were undertaken to determine the geotechnical and physical properties of subsurface materials. Inclinometer casings were installed to enable monitoring of ground movement at approximately 3 monthly intervals. Groundwater levels are monitored at 30 minute intervals with automatic loggers installed in piezometers with short screen intakes at the sand-clay interface. Monitoring between 26th May 2005 and 25th May 2007 captured a 1 in 10 year storm event in June, 2005. Monitoring will continue to observe response to at least three 1 in 10 year storm events.

Monitoring Results

Groundwater response to a major rainfall event

The only significant rainfall event that occurred during the monitoring period was the 385 mm that fell over several days around 30th June, 2005 (250.4 mm maximum daily rainfall). This event was a 1 in 10 Average Return Interval (ARI) event for a 2 hour duration, and 1 in 94 ARI event for an 18 hour duration.

The lower piezometers across the study area (ie. sites B and C) showed greater response to this June, 2005 event than the upper piezometers (Table 2, Figure 3). Sites 1A to 3A showed 0.6 to 0.95 m rise in level over a period of approximately 3 to 7 days. Sites 1B, 1C, 2C, 3B and 3C displayed rises of 1.12 to 1.74 metres over a period of between 12 and 24 hours. However, it is noted that the initial rise rate of piezometer 3A was fast but delayed. The lower piezometers all began to respond to the event before the upper piezometers.

This data reflects the fact that there is a limited catchment area upslope of the study area. As a result, piezometers located at near the base of the slope are characterised by a larger response to rainfall infiltration over a larger, upslope area.

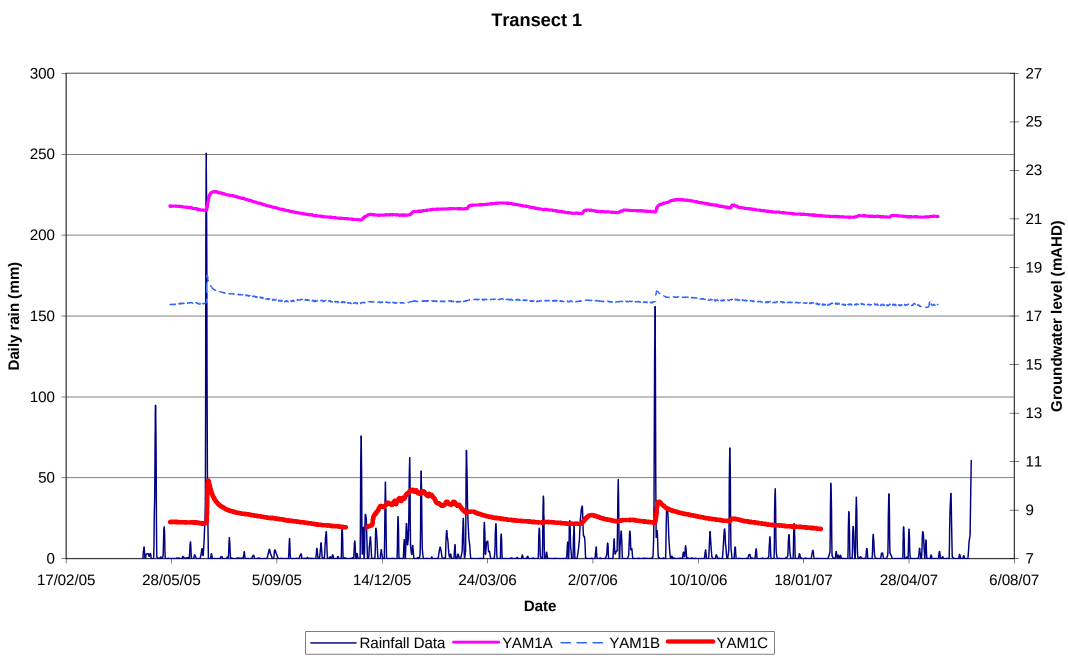

Fig 3 Groundwater levels for Transect 1 (May 2005- May 2007)

Inclinometer monitoring of slope movement

Inclinometer monitoring was carried out during the period of 29 April 2005 (initial baseline setup reading) to 8th March 2007. All inclinometers with the exception of Location 2C indicated little or no significant movement. However, Location 2C showed 5 mm of movement with the plane of movement at a depth of about 3 m. Sandy clays exist at 3 m depth in Borehole 2C, therefore the movement is occurring within the sandy clays.

The mode of movement was consistent with the stability results for Transect 2, which showed that the lowest factor of safety occurs for slip circles at the toe of the slope when higher groundwater levels exist during rainfall periods. The movement in inclinometer 2C tends to suggest a slow moving (Creep) type movement occurring at the toe of the slope.

Table 2
Groundwater Level Response to Rainfall Event 29th June – 7th July 05

Location	Statistics		
	Magnitude of Rise	Lag*	Maximum Rate of Rise
	m	*hours*	*mm/hr*
1A	0.75	176.50	21
1B	1.12	12.00	222
1C	1.74	17.00	58
2A	0.63	116.25	18
2C	1.17	22.75	117
3A	0.95	75.50	219
3B	1.31	9.50	265
3C	1.19	34.25	122

* Lag time for each piezometer is the difference between time of first response and time at which peak levels were recorded.

Updated Assessment for Yamba Hill

Hydrogeological conceptual model

The observed behaviour of groundwater levels in response to rainfall during the monitoring period was consistent with the initial conceptual hydrogeological model (J&K, 2000). Detailed analysis of borehole stratigraphy and groundwater monitoring data was used to verify and refine the conceptual model. Groundwater parameters of relevance to slope stability assessment are summarised in Table 3, comparing the initial model with observations from the site. Observed saturated thickness, depth to groundwater, groundwater level rise, lag times and drainage times were all within the range of values allowed for by the initial hydrogeological model.

In summary, observed conditions were consistent with the simplified hydrogeological model, and observed groundwater level response (0.75 to 1.74 m) to rainfall events was within the range (1 to 2 m) that was predicted. The hydrogeological model that was adopted is therefore considered to be conservative and appropriate for the Yamba Hill site.

However, the hydrogeological model was improved by accounting for a lower rather than higher lateral hydraulic gradient after rainfall events (Figure 4). This result suggests that the toe of the slope becomes saturated during rainfall events. The hydrographs near the toe of slope had two distinctive recovery rates – early rapid decline (ie. 77 mm/day) then slower decline (ie. 6 mm/day) while the average drainage rate was close to the 19 mm/day in the initial conceptual model. The two stage drainage curves were attributed to local drainage immediately after the rain period, followed by a slower groundwater level decline due to additional infiltration from further up the slope arriving at the toe of the slope.

Table 3

Summary of conceptual groundwater models

Parameter	**Initial model (J&K 2000)**	**Revised model (June 2005 event)**
Groundwater level rise (m)	1 to 2	0.75 to 1.74
Saturated zone above silty clay or bedrock (m)	0.5 to 1 m	0.3 to 3.5
Depth of sand at crest of slope (m)	3	3.8 to 16.4
Depth of sand at toe of slope (m)	1 to 2	3.8 to 4.2
Depth to clay (m)	9 to 10	1.7 to 16.4
Thickness of clay (m)	2 to 4	0.2 to 3.3
Vertical flow to crest of slope – time lag between rainfall and groundwater level rise (days)	3	3 to 7.4
Vertical flow to toe of slope – Time lag between rainfall and groundwater level rise (days)	0.7 days	0.7 to 1.4
Lateral flow - time lag between rainfall and groundwater level rise (days)	8 to 40 ^	11 to 67 (Piezo 1C)
Time to drain after event (days)	10 to 80 days (av. 19 mm/day)	96 days (Piezo 1C)
Average rise in groundwater levels for 100 mm event (mm)	375 mm# (300 to 400 mm)	286 mm

^ assuming K lateral is 2 to 10 times vertical K

assumes 75% average permeable area and 0.2 effective porosity

Lateral groundwater flow rates for Transect 1 were calculated to be 0.18 m^3/day similar to the 0.2 m^3/day projected by the initial groundwater model (Figure 4). These flow rates assumed a saturated hydraulic conductivity value of 10^{-5} m/s or 36 mm/hr (Douglas Partners, 1996). These values are considered to be realistic for silty sand, and are unlikely to be improved by site specific testing.

Limited drainage rates mean that groundwater levels below the crest of the dune remain high for 2 to 3 months after a major rainfall event. Therefore, antecedent rainfall will be the most critical element in producing groundwater levels which may cause instability of the slope.

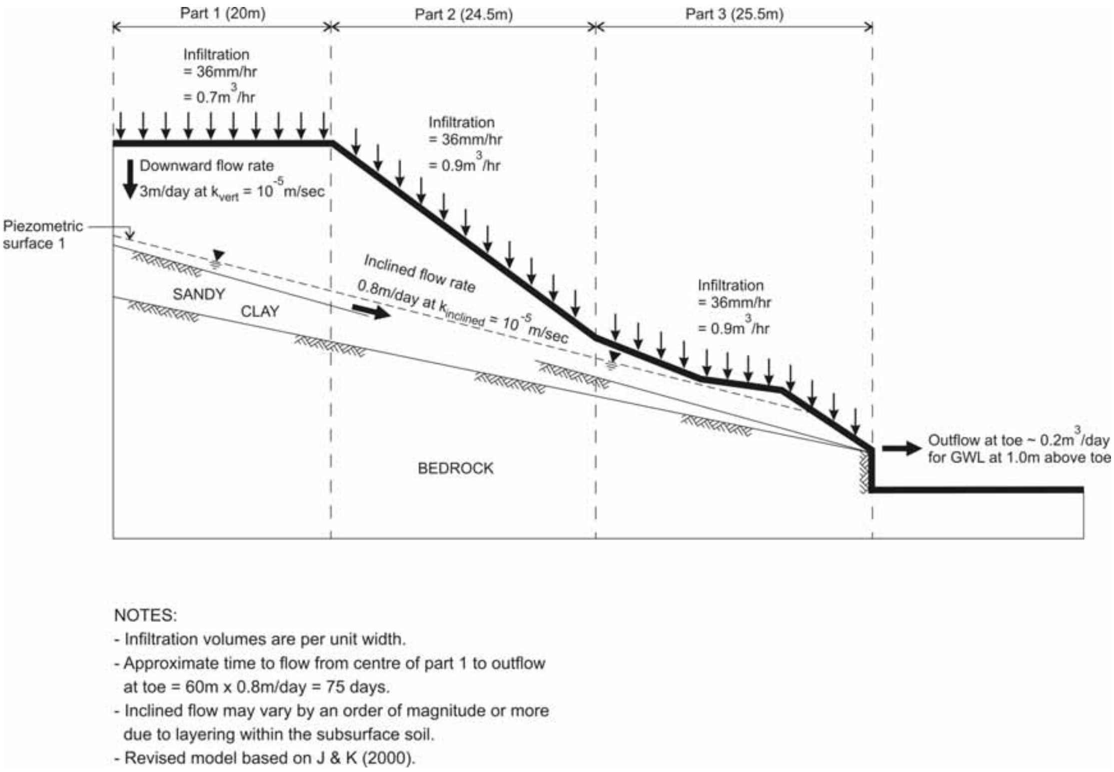

Fig 4 Revised hydrogeological conceptual model (modified from J&K, 2000).

Geotechnical stability analysis

The slope stability analysis considered surface conditions, sub-surface conditions, including suitable material properties (eg. effective cohesion, effective friction angle), together with groundwater level response to rainfall. A computer program SLOPE/W was used to analyse slope stability by considering circular failures through the sandy silt overlying bedrock. Although circular failure is not always the case, it is considered to be a reasonable approximation for many failures.

Overall the stability analysis of three subsurface models (Transects 1, 2 and 3) showed the slopes have low factors of safety (FOS) particularly for higher groundwater levels and the slope close to the Pacific Hotel. Calculated FOS values ranged from 1.0 to 1.6 for varying locations, types of failures and groundwater level conditions. These FOS values were generally less than the usually accepted values of at least 1.5 for reasonable design case and as low as 1.25 that may be tolerated for transient short term conditions.

Coastal processes and hazards

Consideration of coastal processes indicates that hazards are increasing over time at Yamba Beach due to a combination of storm erosion, shoreline recession and long term weathering and erosion of the rock cliffs. Both storm erosion and future recession will erode sand from the toe of the slopes fronting Yamba Beach, however, much of the beach (particularly the northern end) is fronted by rock cliffs/shelves which would limit short term erosion.

The rock cliffs and ledges would resist short term beach erosion, however when exposed, they are likely to suffer from long term weathering and erosion. Detailed studies would need to be undertaken for Yamba Beach, but studies at other sandstone coastlines in NSW indicated mean erosion rates between 1 and 5 mm/year. These reported mean rates need to be interpreted with regard to the episodic nature of cliff collapse events – that is, many years elapse between major events.

Landslide Risk Analysis

The risk analysis included rainfall analysis and probability assessment, in accordance with the AGS (2000) Risk Management Guidelines. The earliest known landslide occurred in May, 1938 with several recorded events since then (J&K 2000), although we are not aware of any landslides during the monitoring period for this project.

The probability of a landslide occurring was determined to be as follows:

- For earthslides and scour at the toe of the slope – 1 in 10 years to 1 in 125 years. However, considering that a landslide occurs only 50% of the time a ‘trigger’ level is reached, the probability equates to $5x10^{-2}$ to $4x10^{-3}$.

- For earthslides encompassing the steeper hillslide slopes – 1 in 10 years to 1 in 100 years. For a 50% trigger, this probability equates to $5x10^{-2}$ to $5x10^{-3}$.

Risk was then determined as a function of probability and consequence. Risk estimates were determined in relation to the suggested criteria in AGS (2000), with 10^{-4} tolerable risk and 10^{-5} as acceptable risk for loss of life of person most at risk. It will be up to the owners to decide whether these values are appropriate and the conclusions regarding the risk estimates reasonable.

The highest risk values identified were associated with Landslide Risk Zone 1a (LRZ1a, Figure 5). This zone was characterised by steepest slopes, a history of movement and expected high occupancy rate. In this zone the results of the risk assessments were:

- For slow to very slow movement $5x10^{-5}$ (tolerable)
- For rapid to very rapid movement 10^{-3} (unacceptable)

For LRZ1b which includes residential dwellings to the north of the Pacific Hotel the risk assessments were:

- For slow to very slow movement 10^{-5} (acceptable, just)
- For rapid to very rapid movement $4x10^{-4}$ (unacceptable)

The data obtained from investigations and monitoring during this project do not allow any adjustment to the LRZs.

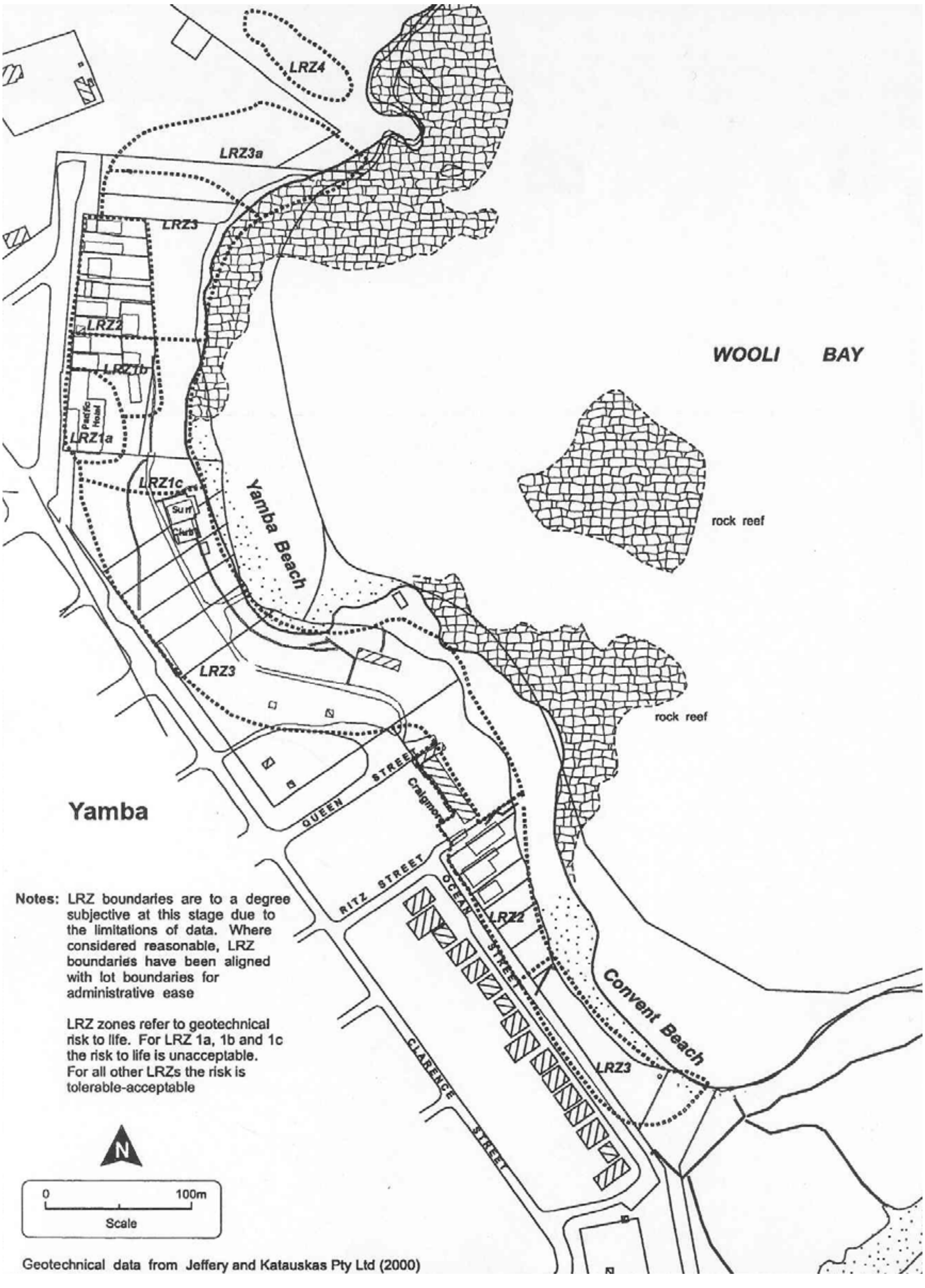

Fig 5 Landslide risk zones (After J&K 2000 and MHL 2003).

Summary and recommendations

In summary, investigations and monitoring at Yamba Hill to date have confirmed previous estimates of groundwater level response to major rainfall events and slope stability assessments. The hydrogeological conditions and recharge response that was observed was within the range expected for a sand dune sediments overlying a weathered sandstone slope, however it is noted that only a single 1 in 10 year ARI event occurred during the 2 year monitoring period.

All inclinometers with the exception of Location 2C indicated little or no significant movement of the hill slope. However, Location 2C showed 5 mm of movement with the plane of movement at a depth of about 3 m. The movement in inclinometer 2C tends to suggest a slow moving (Creep) type movement occurring at the toe of the slope.

On the basis of the revised slope stability risk assessment, it is considered that emergency levels and the subsequent management implications that were put into place in October 2000 should remain in place until more permanent stabilisation measures are adopted.

Two warning levels were set up as an interim measure: an Orange level which was based on a 1 in 3 year rainfall and a Red level which was based on a 1 in 10 year rainfall, taking into account antecedent rainfall over periods of 1 to 90 days. Various slope treatment/stabilisaton options should be investigated in more detail with a view to implementation as soon as possible.

Groundwater monitoring should also continue, and inclinometer measurements extended to an annual basis unless significant rainfall events occur and/or movements of the slope are observed, in which case the inclinometers should be read as soon as possible.

References

AGS (2000). Landslide Risk Management Concepts and Guidelines. Australian Geomechanics Society Sub-Committee on Landslide Risk Management.

CVC (2004). Yamba Coastline Management Plan – Implementation Strategy. Issued 16 March, 2004. Clarence Valley Council

Jeffrey and Katauskas Pty Ltd (2000). Report to Manly Hydraulics Laboratory NSW Department of Public Works and Services on Geotechnical Assessment for Yamba Coastline Management Study.

Jeffrey and Katauskas Pty Ltd (2007). Report to UNSW Water Research Laboratory on Updated Geotechnical Assessment for Pilot Hill, Yamba, Draft September, 2007.

MHL (2003). Yamba Coastline Management Study - Stage 4 - Draft Coastline Management Plan. Report MHL1142, June 2003.

WRL (2007). Yamba Hill Groundwater Level Monitoring Assessment to Mid-2007. UNSW Water Research Laboratory 2007-32 report to Clarence Valley Council, Draft, September, 2007.

WRL (2005). Yamba Hill Groundwater Level Monitoring Interim Report. UNSW Water Research Laboratory Technical Report 2005-34 report to Clarence Valley Council.

APPENDIX E

GROUNDWATER LEVEL DATA MISSING – LOW WATER LEVEL AND LOGGER FAILURE

APPENDIX E

Groundwater level data missing - low water level and logger failure

Piezo	End date	Start date	Days	Dry or failure ?
Yam2A	6/07/2006	13/10/2006	99	failure
Yam2C	7/09/2005	11/01/2006	126	Water level below sensor until 10/10/2005 (9.16) then failure
Yam3B	15/07/2005	2/09/2006	414	Water level below sensor, sensor is down in bore as low as it will go without being in mud at bottom (3.96m)
Yam3B	14/09/2006	25/05/2007	253	Water level below sensor, sensor is down in bore as low as it will go without being in mud at bottom (3.96m)
Yam3C	3/10/2005	27/11/2005	55	Water level below sensor, sensor is down in bore as low as it will go without being in mud at bottom (1.68m)
MSA1	22/10/2005	29/12/2005	68	Water level below sensor, sensor is down in bore as low as it will go without being in mud at bottom (8.60)
MSA1	10/02/2007	25/05/2007	104	Water level below sensor, sensor is down in bore as low as it will go without being in mud at bottom (8.60m)
MSA9	12/11/2005	2/12/2005	20	Water level below sensor, sensor is down in bore as low as it will go without being in mud at bottom (5.44m)
MSA5	all missing			All dry (8.00)