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IPWEA PUBLIC WORKS TRAINING WEEK 2016

WATER SENSITIVE URBAN DESIGN

Stormwater Infiltration Rates

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Abstract

Most drainage networks in Perth have multiple soakwells for stormwater infiltration in both private and public land. The humble soakwell has served local government and private householders well over many years, rarely requiring maintenance and continuing to function close to design specification over many years. The original design drawings or testing, specifying the required size of holes in the base and sides to allow water to pass from the inside of the soakwell through to the soil outside, have long been lost or forgotten.

Stormwater Infiltration Testing (SIT) was performed at a lot scale at Rivergums Estate Baldavis in December 2014 (JDA, 2015).

Stormwater Infiltration Testing (SIT) was also performed on a 1200 × 1200 mm concrete soakwell at a development site in the City of Armadale during September and October 2015, with high water table, and used a water tanker rather than rainfall runoff for the water source.

This empirical testing of a soakwell is the first to have been documented, so far as the authors are aware. A new, West Australian developed infiltration device "Tunnelwell®" (Tunnelwell) was also subjected to infiltration testing and results are included in this paper.

The tests show high infiltration rates into the underlying sand. These would equate to high continuing losses from rainfall, which would consequently reduce the volume of rainfall that becomes stormwater runoff.

It is hoped that these results will be incorporated in future stormwater manuals produced by metropolitan local authorities.

1. What are soakwells?

A soakwell is a vertical cylinder with base and side holes, usually made from concrete but in some cases from plastic that collects stormwater and infiltrates it into the ground. They are extensively used in Western Australia, where the sandy soils allow for water to be infiltrated 'at source' into the groundwater table.

Soakwells in West Australia (WA) are installed both on housing lots, commercial and industrial estates, and in road reserves.

2. What is Tunnelwell?

Tunnelwell is a WA designed and built stormwater infiltration device. It has optimized sidewall opening (louvres) which prevent sand washing in, and precludes the need for geotextile wrapping which may get clogged over time. Tunnelwell can be laid directly on sand without a base, thus increasing the area available for infiltration.

3. Infiltration testing at Rivergums, Baldivis (December 2014)

3.1 Introduction

JDA was appointed by Cedar Woods to conduct a rainfall runoff experiment, by applying water from water tankers, on a display home in the Rivergums development in Baldivis WA, on the corner of Bonney Drive and Como Way (see Figure 1). Water table elevation at the lot is estimated to vary seasonally between 4.30 and 4.80 mAHD pre-development (RPS, 2013).

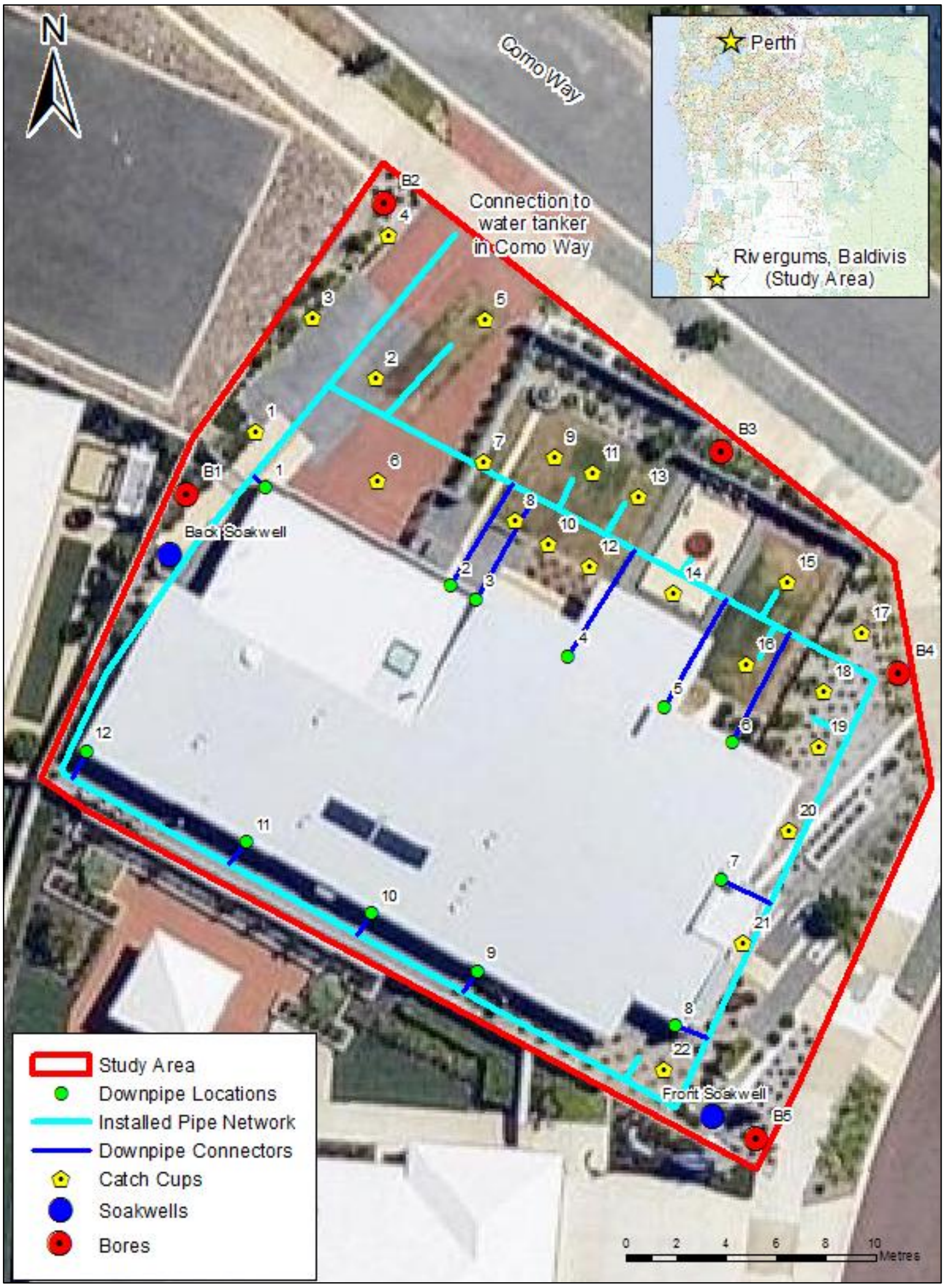

Figure 1: Location Plan and Experimental Layout

The Rivergums is a residential development in Baldivis, located approximately 40 km south of Perth. The lot used for this experiment is situated on the corner of Bonney Drive and Como Way, with a block size of 787 m². The house on the site, the Urban Retreat, was designed and built by the Rural Building Co. Drawings provided to JDA by the Rural Building Co were used to design an appropriate water application system on the lot to simulate a storm event.

Stormwater drainage for Rivergums was designed by TABEC Consulting Engineers, who provided engineering drawings to JDA showing stormwater, detailing side entry pits, pipe locations and road elevation.

The predominant geological setting underlying the study area is comprised of sand over limestone. The environmental geology of the area is indicative of Bassendean Sands, light grey and fine to medium grained. (Ref: Perth 1:50,000 Environmental Geology MAP). RPS (2013) conducted a geotechnical survey of the surrounding area and found it to be primarily comprised of the Bassendean Sand formation, with large expanses of the Guilford Clay formation.

The hydraulic conductivity of the Bassendean Sands formation is about 15 m/day, however it can range from 10 to 30 m/day and the specific yield is about 0.20 (Davidson, 2006).

JDA proposed to simulate a 10 year ARI 12 hour storm event, according to the 1987 Australian Rainfall and Runoff IFD data for Perth (EA, 1987).

3.2 Experimental Setup

The various proportions of each land cover type present on the lot was analysed in order to determine the volume of flow necessary for each portion of the site. For instance, the roof area was calculated to be approximately 350 m², representing 45% of the total lot area of 787 m² (see Figures 1 and Table 1).

TABLE 1: SUMMARY TABLE ON 12 HOUR 10 YEAR ARI STORM OVER 787M2

Time Step	Time	%	∑ %	depth (mm)	L/30 mins	L/s	Roof (L)	Roof (L/s)	Balance (L)	Balance (L/s)	Each Downpipe (L)	Each Downpipe (L/s)
1	0.5h	13.8	13.8	9.1	7125	4.0	3206	1.8	3919	2.2	267	0.15
2	1h	27.0	40.8	17.7	13939	7.7	6273	3.5	7667	4.3	523	0.29
3	1.5h	8.5	49.3	5.6	4388	2.4	1975	1.1	2414	1.3	165	0.09
4	2h	4.3	53.6	2.8	2220	1.2	999	0.6	1221	0.7	83	0.05
5	2.5h	6.7	60.3	4.4	3459	1.9	1557	0.9	1902	1.1	130	0.07
6	3h	5.5	65.8	3.6	2839	1.6	1278	0.7	1562	0.9	106	0.06
7	3.5h	4.2	70.0	2.8	2168	1.2	976	0.5	1193	0.7	81	0.05
8	4h	4.9	74.9	3.2	2530	1.4	1138	0.6	1391	0.8	95	0.05
9	4.5h	3.7	78.6	2.4	1910	1.1	860	0.5	1051	0.6	72	0.04
10	5h	1.6	80.2	1.0	826	0.5	372	0.2	454	0.3	31	0.02
11	5.5h	1.8	82.0	1.2	929	0.5	418	0.2	511	0.3	35	0.02
12	6h	1.4	83.4	0.9	723	0.4	325	0.2	398	0.2	27	0.02
13	6.5h	3.1	86.5	2.0	1600	0.9	720	0.4	880	0.5	60	0.03
14	7h	2.7	89.2	1.8	1394	0.8	627	0.3	767	0.4	52	0.03
15	7.5h	2.3	91.5	1.5	1187	0.7	534	0.3	653	0.4	45	0.02
16	8h	2.0	93.5	1.3	1033	0.6	465	0.3	568	0.3	39	0.02
17	8.5h	1.2	94.7	0.8	620	0.3	279	0.2	341	0.2	23	0.01
18	9h	0.9	95.6	0.6	465	0.3	209	0.1	256	0.1	17	0.01
19	9.5h	0.4	96.0	0.3	207	0.1	93	0.1	114	0.1	8	0.00
20	10h	1.1	97.1	0.7	568	0.3	256	0.1	312	0.2	21	0.01
21	10.5h	1.0	98.1	0.7	516	0.3	232	0.1	284	0.2	19	0.01
22	11h	0.7	98.8	0.5	361	0.2	163	0.1	199	0.1	14	0.01
23	11.5h	0.7	99.5	0.5	361	0.2	163	0.1	199	0.1	14	0.01
24	12h	0.5	100.0	0.3	258	0.1	116	0.1	142	0.1	10	0.01
	Average			2.7	2151	1.2	968	0.5	1183	0.7	81	0.04
	Total	100		65.6	51627		23232	12.9	28395	15.8	1936	1.08
	12 hr 10yr storm = 65.6 mm				Roof Area = 350 m^2 (45% of lot)					12 Downpipes		
	787m^2 * 0.0656 m = 51.7 m^3				Balance Area = 437 m^2 (55% of lot)							

The remaining 437 m² representing 55% of the lot comprises a mixture of impervious and pervious surfaces; with the vast majority being classified as pervious.

There are 12 downpipes from the roof of the home entering into a piped network and subsequently into two 1500 x 1500 mm soakwells (see Figure 1).

Five downpipes (numbered 5 to 9) are connected to the front soakwell: seven downpipes (numbered 1 to 4 and 10 to 12) are connected to the rear soakwell.

The temporal distribution of rainfall through the 12 hours of the 10 year ARI event to be simulated was taken from EA (1987). The temporal pattern has 24 time steps each of 0.5 hours duration with a large proportion of the total rainfall in the first 2.5 hours of the storm (see Table 1 & Figure 2).

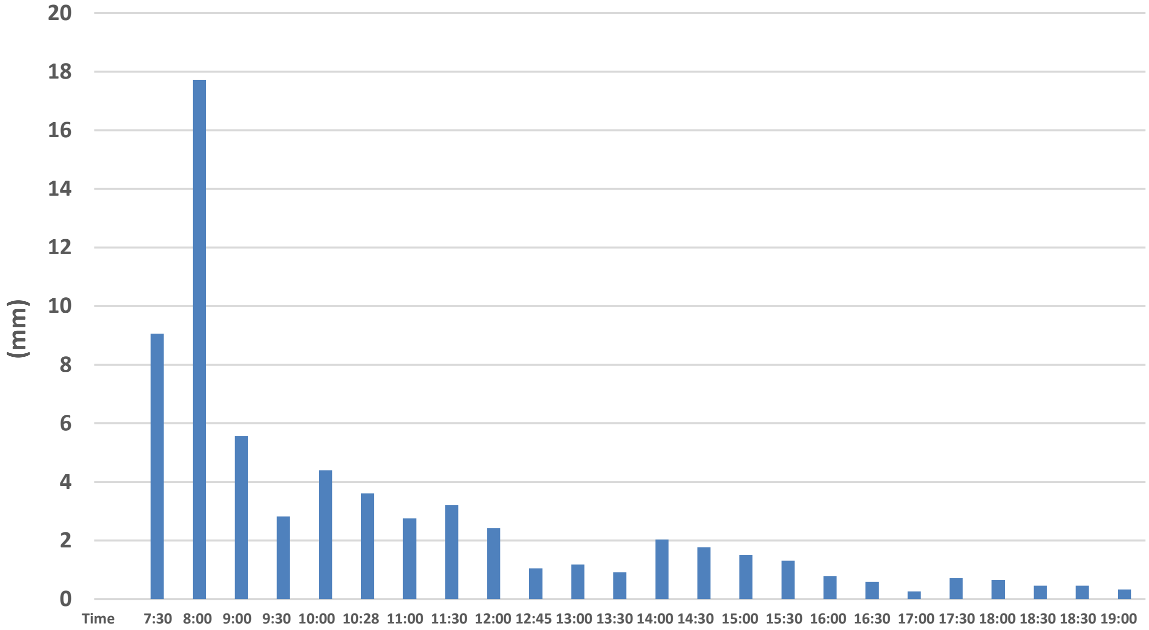

Figure 2: Design Rainfall

In particular the temporal pattern has 13.8 % of the total rainfall occurring in the first 0.5 hour time step, and a further 27 % during the second 0.5 hour time. After this time, the percentage of the total rainfall in each time step greatly reduces.

To simulate a 12 hour 10 year ARI storm event it was necessary to determine the total volume of water required. This was calculated using the total lot area, multiplied by the total depth of water applied during the storm event as given by the IFD calculation. The calculation was as follows:

12 hour 10 year ARI storm = 65.6 mm

787 m^2 x 0.0656 m = 51.7 m^3

This calculation indicated that 51.7 m^3 of water should be applied to simulate the storm event (see Table 1).

In order to apply the correct volume of water to represent the 12 hour 10 year ARI storm, a 50 mm PVC pipe network was temporally installed aboveground connected to a 15,000 L water tanker in Como Way (see Figure 1).

The pipe work (see Figure 1) had 19 sprinklers attached (not shown on Figure 1), each capable of pumping a maximum of 9.1 L/minute. In addition 15 mm diameter pipes were run from the 50 mm line to each of the 12 roof downpipes to simulate an appropriate volume of water captured by the roof during the storm, as shown on Figure 1.

The flow rate to each downpipe was regulated every half hour to reproduce the appropriate temporal pattern on the roof. The total volume applied to the downpipes was distributed equally between them.

Five monitoring bores were hand augered on 22/12/14 by JDA into the water table on the lot to allow measurement of groundwater levels during and after the experiment to see the extent of groundwater mounding and on the lot generally and specifically associated with the front and rear soakwells (see Figure 1).

The lot contained both lawn area and plants which are irrigated from the mains supply on irrigation days of Tuesday, Friday and Sunday.

The irrigation was turned off by JDA on the evening of Monday 22/12/14 so that no irrigation was applied during the day of the experiment Tuesday 23/12/14. At 5 pm on Tuesday 23/12/14 the irrigation controllers were switched back on so that irrigation continued on the set days thereafter.

The intention was to measure any runoff from the lot onto the roads by lifting the lid of adjacent side entry pits and measuring the rate of flow into the pits using a calibrated 10 L container.

3.3 Monitoring Data

3.3.1 Flow Rate

Two 15,000 L water tankers were engaged for the experiment:

- Both tankers arrived at site full at 7.00am 23/12/14.
- When the first tanker was nearly empty it was refilled from the second which then refilled from the nearest street hydrant.
- The water tankers were both fitted with flowmeters (cumulative and actual) and pressure gauges.

Throughout the experiment the pressure reading was approximately 100 kPa and the total volume of water applied as measured by the tanker flowmeters matched the initial condition of 2 full tankers plus the difference in reading on the Water Corporation stand pipe.

For instance, in the first 30 minutes of the storm simulation it was calculated that 7419 L would be required, and a further 14515 L in the second 30 minute time step. This was equal to flow rates of approximately 4.1 and 8.1 L/s respectively. The flow rate and total litres used throughout the experiment are shown in Figure 3.

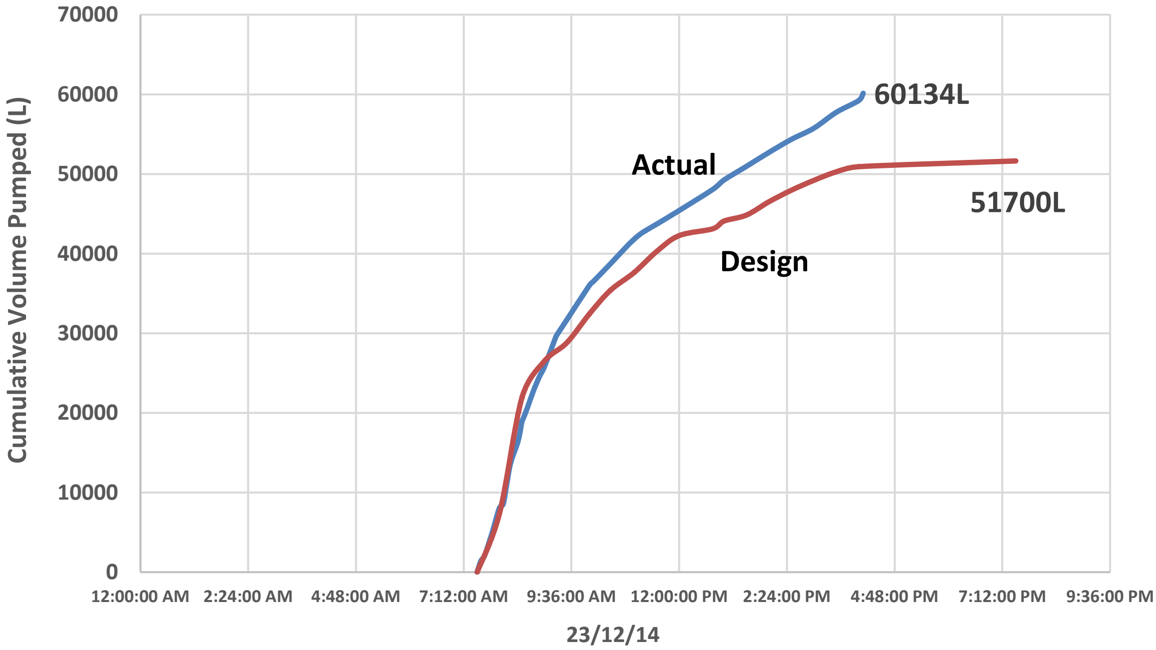

Figure 3: Actual vs Design Cumulative Volume Pumped (L)

Figure 3 illustrates a comparison between the applied pumping rate in L/s and the design pumping rate, also in L/s. As can be seen, the maximum required pumping rate was 8.10 L/s during the second 0.5 hours interval of the test, however the maximum applied rate at this time was 7.16 L/s (Figure 4). This maximum rate exceeded the tanker pump capacity and therefore the application rate was held constant at approximately 7 L/s for a longer period of time to present the correct volume.

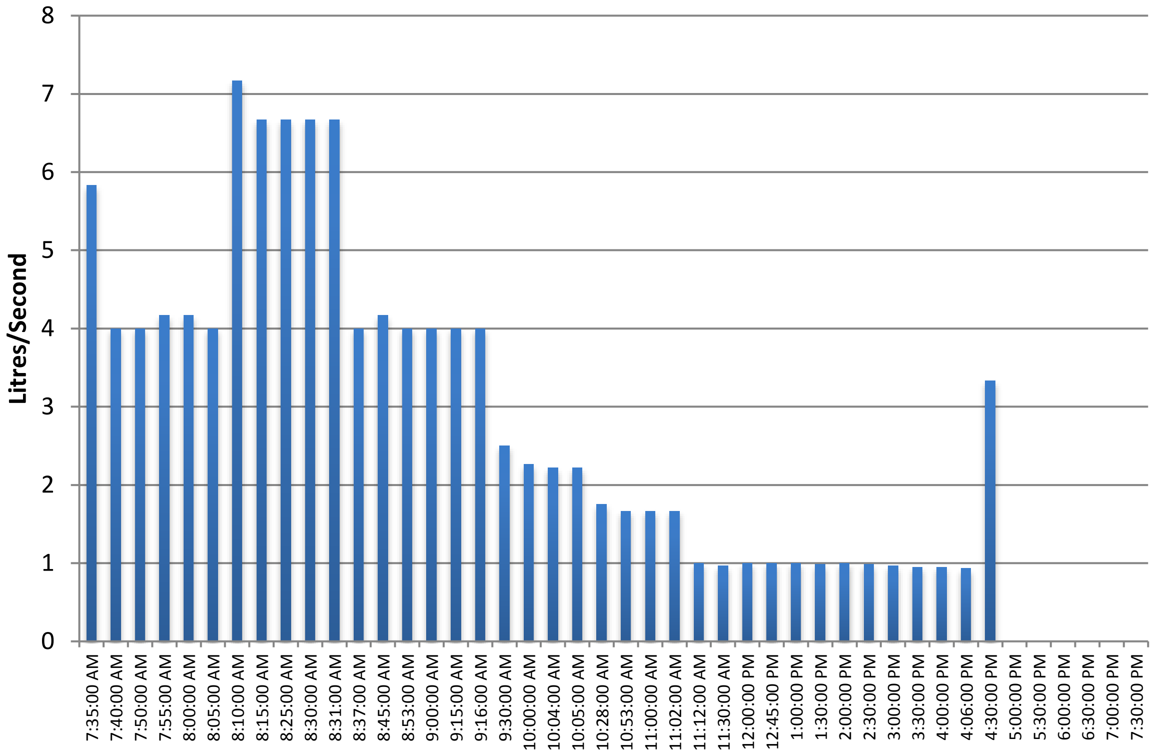

Figure 4: Actual Flow Rate (Litres/Second)

After this initial period, the two curves began to stabilise and at many points were very similar; for instance a value of 1.66 L/s was recorded at 10:30 am compared with 1.60 L/s according to the temporal pattern.

Towards the end of the experiment, the water application rate was maintained higher than the temporal pattern to make full use of the available volume in the water tankers.

Figure 3 illustrates the cumulative volume pumped through the system (litres) in terms of the design storm and the actual applied volume. As this figure illustrates, the initial stages of the test followed the design temporal pattern closely, however after the first 2 hours, the applied rate exceeded the pattern.

In total 60,134 L were applied compared with 51,700 L required for the 12 hour duration 10 year ARI storm.

Due to the low application rates required in the final 3 hours of the storm, the experiment was terminated after 9 hours at 4.30pm on 23/12/14.

Table 2 below describes the actual recorded pumping rate throughout the experiment.

TABLE 2: WATER TANKER METER READINGS

Time (Hrs)	(LPM)	Total (L)
7:30	350	0
7:35	240	1400
7:40	240	2090
7:50	250	4930
7:55	250	6590
8:00	240	8100
8:05	430	8600
8:10	400	11300
8:15	400	13790
8:25	400	16550
8:30	400	19000
8:31	240	19150
8:37	250	20600
8:45	240	22700
8:53	240	24500
9:00	240	25800
9:15	240	29500
9:16	150	29700
9:30	136	31700

Time (Hrs)	(LPM)	Total (L)
10:00	133	36000
10:04	133	36400
10:05	105	36470
10:28	100	38700
10:53	100	41200
11:00	100	41800
11:02	60	42000
11:12	58	42700
11:30	60	43700
12:00	60	45406
12:45	60	48060
13:00	59	49238
13:30	60	50960
14:00	59	52700
14:30	58	54340
15:00	57	55738
15:30	57	55778
16:00	56	59215
16:06	200	60134

3.3.2 Groundwater Levels

During the experiment, groundwater levels were measured in each of the 5 monitoring bores on the lot after every 30 minute interval. A measurement was also taken at each of the bores prior to the commencement of the experiment in order to gain a base value against which subsequent levels could be compared. The experiment began at 7:30 am (23/12/14), and the last readings were taken around 4:00 pm (23/12/14). The information gathered is presented below in Table 3 and also in Figure 5.

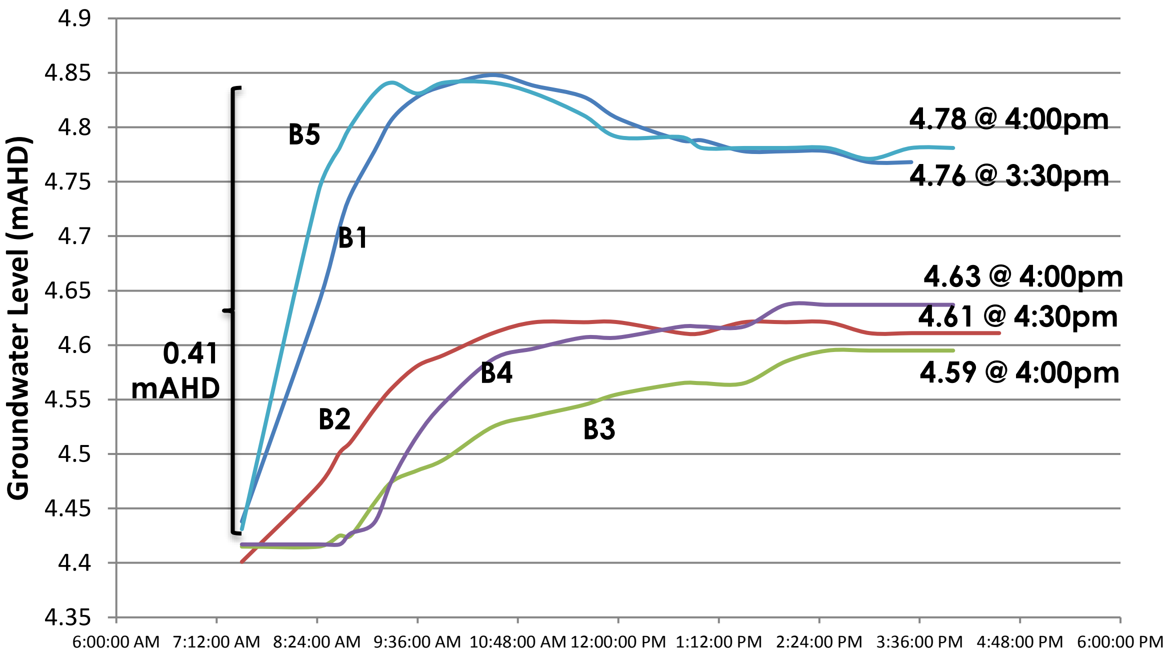

Figure 5: Combined Data (23/12/14)

TABLE 3: RECORDED GROUNDWATER LEVELS 23/12/14

Time (Hrs)	Groundwater Levels (mAHD)				
	B1	B2	B3	B4	B5
7:30	4.43	4.40	4.41	4.41	4.43
8:25	4.63	4.47	4.41	4.41	4.74
8:40	4.70	4.50	4.42	4.41	4.78
8:48	4.73	4.51	4.42	4.42	4.80
9:05	4.77	4.54	4.45	4.43	4.83
9:18	4.80	4.56	4.47	4.47	4.84
9:36	4.82	4.58	4.48	4.51	4.83
9:55	4.83	4.59	4.49	4.54	4.84
10:30	4.84	4.61	4.52	4.58	4.84
11:00	4.83	4.62	4.53	4.59	4.83
11:35	4.82	4.62	4.54	4.60	4.81
12:00	4.80	4.62	4.55	4.60	4.79
12:45	4.78	4.61	4.56	4.61	4.79
13:00	4.78	4.61	4.56	4.61	4.78
13:30	4.77	4.62	4.56	4.61	4.78
14:00	4.77	4.62	4.58	4.63	4.78
14:30	4.77	4.62	4.59	4.63	4.78
15:00	4.76	4.61	4.59	4.63	4.77
15:30	4.76	4.61	4.59	4.63	4.78
16:00	NA	4.61	4.59	4.63	4.78
16:33	NA	4.61	NA	NA	NA
16:42	NA	NA	NA	NA	NA
16:53	NA	NA	NA	4.62	NA
16:55	NA	NA	NA	NA	4.78
17:00	NA	NA	NA	NA	NA

NA: No readings taken during this time due to logger installation

Figure 6 for bores B1 to B5 over the period 23/12/14 to 9/1/15 shows evidence of spikes in the groundwater level recessions during scheduled irrigation days of Tuesday, Friday and Sunday as noted in Section 3.3 above.

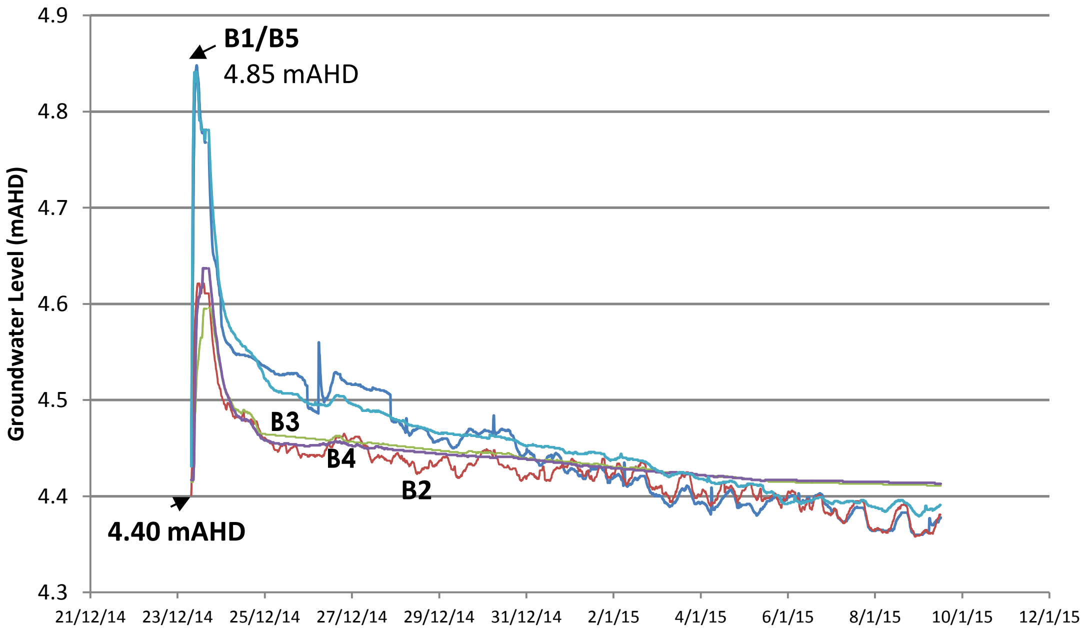

Figure 6: Combined Data all Bores (23/12/14 -19/1/15)

Figures 7 and 8 show groundwater contours at 10:30 am during the day of the experiment Tuesday 23/12/14, and 2 days later on Thursday 25/12/14 respectively, from data logger data.

Figure 7: Groundwater Levels (10:30am 23/12/14)

Figure 8: Groundwater Levels (10:30am 25/12/14)

Contours on both Figures are based on the water levels recorded in bores B1 to B5, and the observation at the front soakwell was full (6.0 mAHD). Back soakwell assumed full also.

Figure 7 shows JDAs interpretation of 0.1 m interval contours corresponding to highest level measured levels with localised groundwater mounds around the front and rear soakwells.

Figures 5 and 7 show the contours as drawn imply maximum 0.6 m water table rise from 4.4 to 5.0 mAHD beneath the house itself. The 3 monitoring bores along the Como Way frontage (B2 to B4) showed water levels of between 4.55 and 4.61 mAHD, suggesting a rise of between 0.15 and 0.2 m.

Figure 8 shows bore water levels having returned to approximately 4.4 mAHD, showing that the mound associated with the applied water had subsided by early January 2015.

Immediately after completion of the experiment at approximately 5:00 pm 23/12/14, a water level data logger was installed into each of the 5 bores, to monitor the groundwater recession following the experiment.

All monitor bores (1 to 5) was surveyed on 9/1/15 relative to a datum nail on Bonney Drive (10.00 m above survey datum, 6.02 m AHD).

The initial groundwater level at the start of the experiment was 4.4 mAHD.

Figure 5 shows water levels in all bores rose during the experiment by up to 0.41 m in B1 and B5 closest to the rear and front soakwells respectively.

Peak water level at bores B1 and B5 was reached during the early part of the experiment, after which water levels fell. Water levels in bores B2 to B4, further from the soakwells, continued to rise throughout the experiment.

Figure 6 show that all bore water levels receded to the pre-experiment groundwater levels approximately by 5/1/15.

3.3.3 Catch Cups

Twenty two 15 mm capacity catch cups were placed in strategic locations across the lot in order to measure the rainfall intensity over a 30 minute period (see Figure 1) These catch cups were recorded on three separate occasions, 8:20 am, 8:45 am, and 9:15 am (see Table 4). No further measurements were recorded after this time as the temporal pattern of the storm fell away significantly, meaning that no usable intensity of rainfall could be recorded in only 30 minutes.

TABLE 4: CATCH CUP DATA (mm)

Cup #	Time/(mm)			TOTAL
	7:30 to 8:20 am	8:20 to 8:45 am	8:45 to 9:15 am	
1	11	7	11	29
2	8	10	11	29
3	16	15	15	46
4	16	15	16	47
5	16	16	16	48
6	16	16	16	48
7	12	15	15	42
8	16	16	16	48
9	16	16	16	48
10	15	14	15	44
11	16	16	16	48
12	15	15	9	39
13	15	15	11	41
14	14	13	9	36
15	16	16	16	48
16	17	16	16	49
17		13	16	29
18		16	16	32
19		16	16	32
Average	14.7	14.5	14.3	41.2

The average catch in the 22 cups between start of the storm (7:30am) and (9:15am) was 41.2 mm compared with the design rainfall of 36 mm and applied rainfall of 35 mm.

This is a further indication that the applied water was of the correct amount and also shows reasonable accurate spatial distribution using the irrigation sprinklers.

3.3.4 Flow from Lot to Roads

Throughout the experiment no surface water flowed from the lot onto Como Way or Bonney Drive road verges or road pavements.

3.4 Results

3.4.1 Water Balance

Of the 60134 L applied the following can be stated:

- There was no surface runoff from the lot to adjacent lots or to the road reserve;
- Taking average December evaporation rate of 5 mm/d the evaporation loss will be approximately 2,000 L over the 437 m2 of the lot which is not roof area;
- The balance (97%) of the applied water (58134 L) infiltrated into the ground and either became recharge into the water table or increased the soil moisture content of the unsaturated zone.

3.4.2 Specific Yield Calculation

The average water table rise as indicated on Figure 7 beneath both the lot itself, adjacent lots and road reserve is approximately 0.2 m over an area of 787 m^2. This corresponds to a specific yield of 0.2.

3.4.3 Runoff Coefficients

As stated above there was no flow from the lot to either Como Way or Bonney Drive reserves. This means that the both the Rational Method and the volumetric runoff coefficient for the 10 year ARI 12 hour storm event for this lot was zero.

Figure 9 shows a graph of 10 year ARI runoff coefficient for the Rational Method of flood estimation plotted against impervious fraction from EA (1987), as a function of the 10 year ARI 1 hour duration rainfall intensity. Two lines are shown corresponding to 25 mm/hr and 70 mm/hr for this storm.

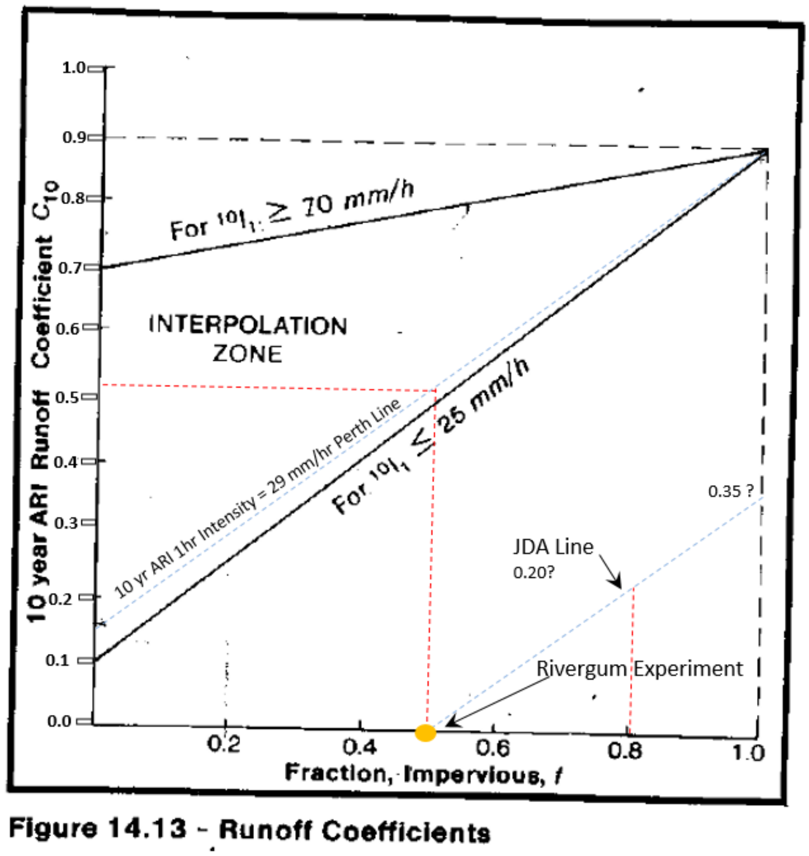

Figure 9: Runoff coefficients (Australian Rainfall & Runoff, 1987)

For the impervious fraction of the subject lot (comprising roof area and other paved surfaces) namely 50%, the Rational Method runoff coefficient from this graph would be estimated as 0.50, compared with the zero value obtained from the experiment.

JDA has annotated an additional line on Figure 9, parallel to the EA line but passing through the data point relating to this experiment. This line has been annotated "JDA Line" for ease of reference.

The JDA Line suggests that with a 100% imperviousness, for the particular conditions of soakwells, soil permeability and depth to water table on this lot, the 10 year ARI runoff coefficient would be 0.35.

It is not possible to measure this by experimentation as there is no such lot on Rivergums Estate with 100% imperviousness. The adjacent carpark represents a 100% impervious site but the runoff from this is connected through to the Como Way storm water system rather than soakwells and hence could not be used as a future experimental site.

For a housing lot with say 80% impervious area, the JDA Line on Figure 9 suggests a Rational Method runoff coefficient of 0.20, corresponding to volumetric runoff coefficient of say 0.25.

3.4.4 Effectiveness of Soakwells

The soakwells front and rear are 1.5 m diameter and 1.5 m deep each with a volume of 2.65 m^3, a total of 5.30 m^3.

Dividing this by the impervious site area suggests they were designed from 15 mm rainfall event from the impervious areas which is fairly typical for metropolitan councils.

In reality the 2 soakwells catered for 65.6 mm of rain rather than 15 mm. This can be understood as 15 mm filling the soakwells themselves and the balance (51 mm) infiltrating to the water table or increasing soil moisture above the water table as described above.

The data in this experiment indicates that a front and rear soakwell on this soil type with approximately 1.8 m clearance to water table (from 6.2 mAHD finished level to 4.4 mAHD water table) will allow disposal of storm water from paved areas with no surcharge of soakwells and no overland flow areas to the lot boundaries, or the road reserves.

3.5 Summary

Using two 15,000 L water tankers, one to refill the other, the 10 year ARI 12 hour duration storm was successfully applied to the lot on 23/12/14 by distributing the water into roof downpipe grates leading to front and rear soakwells, and by irrigation sprinklers attached to a 50 mm PVC main temporally installed above ground fed from the water tanker;

The front and rear soakwells, designed to cater for 15 mm runoff from the imperious area, were able to infiltrate the entire roof runoff area rainfall of 65.6 mm without surcharge;

Groundwater levels as measured in 5 monitoring bores showed water table rises of up to 0.5 m adjacent to the soakwells which is consistent with the total volume applied water having become recharge to the water table and increased soil moisture above the water table;

The groundwater levels receded back to pre-storm levels within 2 weeks;

There was no discharge of the applied water from the lot to the adjacent road reserve, equating to a Rational Method and a volumetric rainfall runoff coefficient of zero;

Measured Rational Method runoff coefficient of zero compares with a Rational Method coefficient estimated using EA (1987) Design Chart for 10 year ARI of 0.5 (Figure 9);

Hence EA (1987) the Design Chart grossly overestimates the Rational Method runoff coefficient for this lot which is characterised by sandy soil, front and rear soakwells designed for 15 mm of rain and 1.8 m depth to water table during the storm event;

Extrapolation of the results of this experiment using the JDA Line on Figure 9 suggest that for a housing lot with 80% impervious area that the Rational Method runoff coefficient would be 0.20, and the corresponding volumetric runoff coefficient perhaps 0.25;

Repeating this experiment on a housing lot with higher impervious fraction would be a useful exercise and the results would need to be interpreted in terms of the conditions of soil type, soakwell size, depth to water table as compared with the Rivergums experiment described in this report.

4 Infiltration testing at Harrisdale Green, City of Armadale (Sep – Nov 2015)

4.1 Introdpuction

An experiment was set up on a sand-filled vacant commercial lot at Harrisdale Green, City of Armadale (Figure 10) to assess the ability of soakwells and Tunnelwell to infiltrate stormwater.

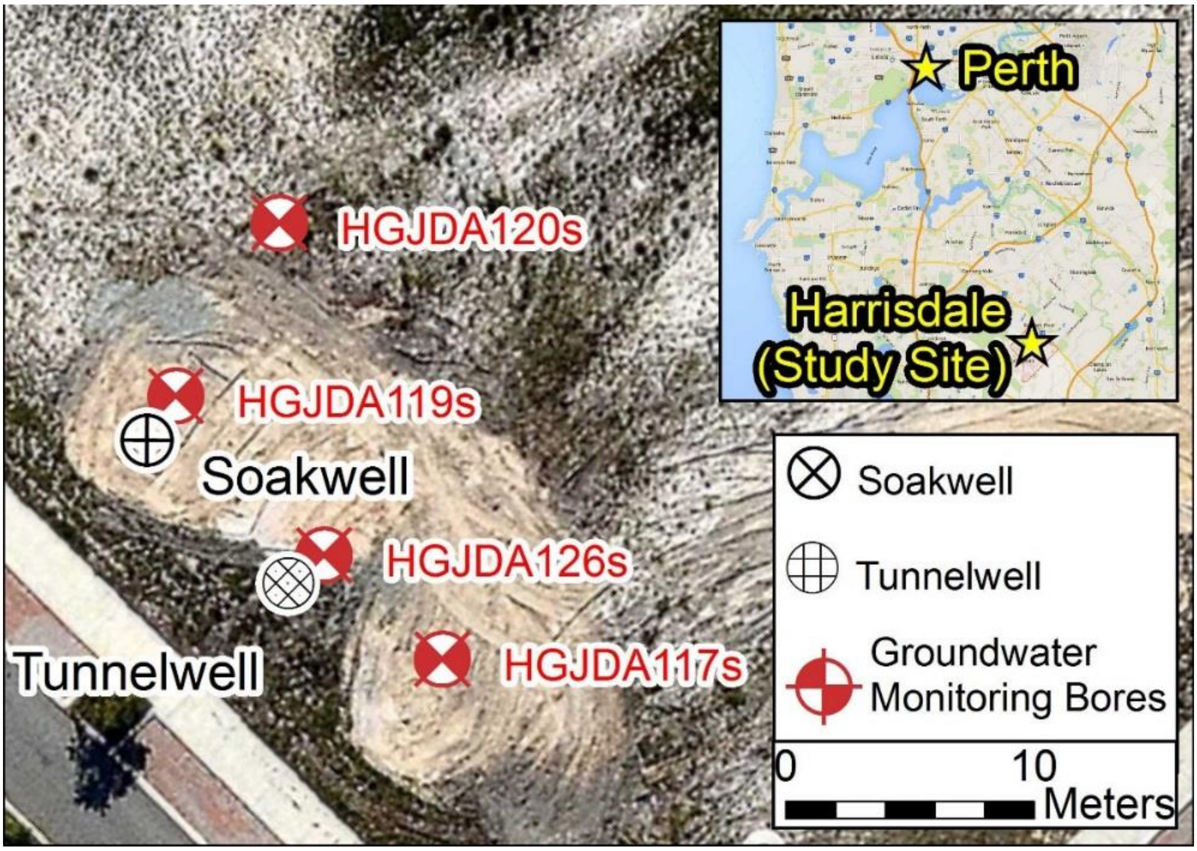

Figure 10: Experimental layout of soakwell, Tunnellwell and monitoring bores.

No such testing appears to have been reported previously, and we are not aware of any data on infiltration rates from soakwells. Infiltration rate depends on soil hydraulic conductivity, interaction with groundwater, as well as on geometry and open areas of the infiltration device. Infiltration is a mirror image of groundwater abstraction, and similar saturated flow equations apply. This experiment is thought to be the first of its kind to collect real data on the performance of infiltration devices.

Tunnelwell is a new product and was tested to see how it compares with a conventional soakwell.

Runoff coefficients for housing lots in similar settings can be derived from these results to assist in sizing of stormwater detention and retention basins.

4.2 Experimental Setup

The site has sandy soil of approximately 2.5 m thickness over poorly permeable cemented sand ("coffee rock").

The experiments were performed during September to November 2015 with water from tanker applied at a uniform rate. The water table varied between the seasonal maximum of 1.16 m below ground level in September, to 1.46 m below ground level in November 2015.

A 1200 × 1200 mm (diameter × depth) soakwell, volume 1.4 m^3, was installed in September 2015, see Plates 1 & 2, together with 2 shallow groundwater monitoring bores at varying distances from the soakwell, see Figure 10 and Plates 1 & 2. This allowed monitoring of the water table mound as water in the soakwell infiltrated into the soil.

Plates 1 & 2: Soakwell on concrete base lowered into excavated hole and wrapped in geotextile fabric, then backfilled to surface.

The first test SIT 1 (15/09/15) was run with no blockages of the soakwell, allowing water to infiltrate through both the slots in the sides and the 0.2 m diameter hole in the bottom.

A second test SIT 2 (12/10/15) was conducted with the hole in the concrete base of the soakwell completely blocked (simulating clogging by foreign objects) so water could only flow out via the slots in the side of soakwell.

A third test SIT 3 (23/11/15) was run with a 1.0 m^2 volume Tunnelwell infiltration device, with blank end caps, giving total volume of 1.4 m^3, the same void capacity as the 1200 x 1200 mm soakwell, see Plates 3 & 4. Two additional monitoring bores were installed to monitor the groundwater mound.

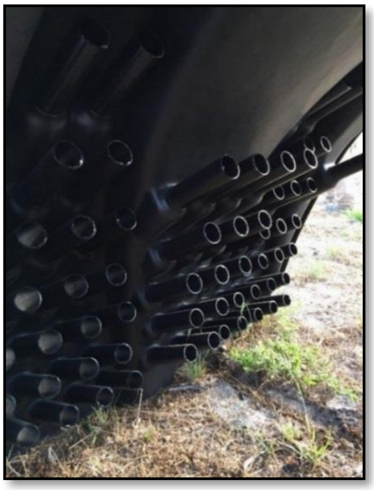

Plate 3: Inside view of the Tunnelwell arch

Plate 4: Tunnelwell in pit ready for backfilling.

In each test SIT 1 to SIT 3, a volume of 15 kL of water from a tanker was discharged into the soakwell and Tunnelwell at a constant rate (1 L/s), rather than simulating a specific storm temporal pattern. Figure 11 shows an IFD graph with the simulated "storm" superimposed showing that the storm ARI increased with duration through the experiment from less than 1 year ARI at 0.1 hours, to greater than 100 year ARI at durations greater than 2 hours.

A typical WA local authority criterion is that soakwells should be sized for 13 mm initial loss. The soakwell and Tunnelwell volume of 1.4 m^3 corresponds to 1.4/0.013 = 105 m^2 impervious area. The total storm runoff volume (15 m^3/105 m^2) = 143 mm.

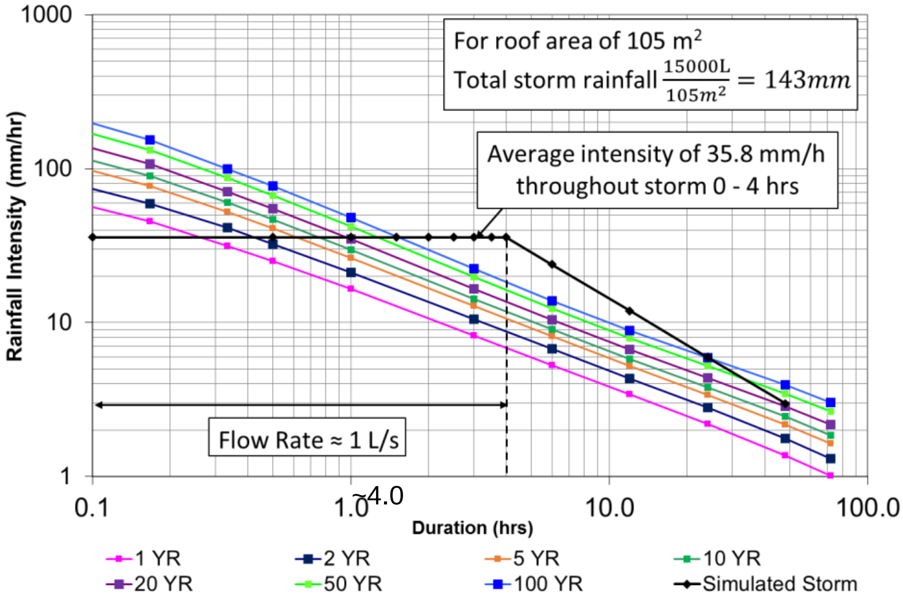

Figure 11: Simulated "storm" (SIT 1 – SIT 3) superimposed on IFD graph.

4.3 Results

In SIT 1-3, 15 kL of water was discharged into the soakwell and Tunnelwell over 4-5 hrs, with both completely filling with water after 1-2 hours in all scenarios. Water that overflowed infiltrated into the adjacent sandy soil.

Figure 12 shows groundwater levels at bores 2 m away from the soakwell and Tunnelwell. All tests showed a similar temporary rise in the local groundwater level followed by a subsequent fall in groundwater levels after the "storm" ended, with water levels on 15 September 2015 (SIT 1) higher than SIT 2 due to a higher initial water table. Though the Tunnelwell test (SIT 3) had the lowest initial water table, it had the highest peak water level, indicating greater infiltration through side wall louvres.

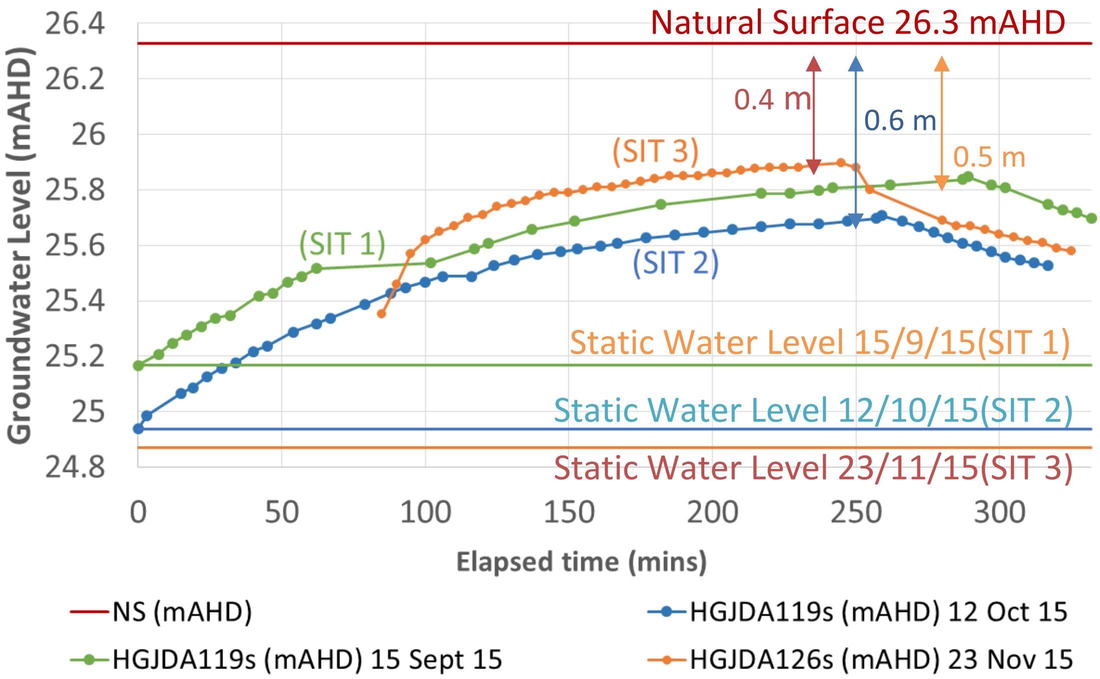

Figure 12: Time series of groundwater levels in monitoring bore 2m way from infiltration experiment.

4.4 Development of recharge cones

As the tests progressed, cones of wetted soil developed, eventually reaching a steady state.

Aquifer transmissivity (T) above the coffee rock layer was estimated from a Theis Equation analysis as 26 m^2/d, corresponding to hydraulic conductivity (K) of 12 m/d and saturated thickness (D) of 2.2 m.

Figure 13 shows cross sections of the soakwell and Tunnelwell and the corresponding recharge cones.

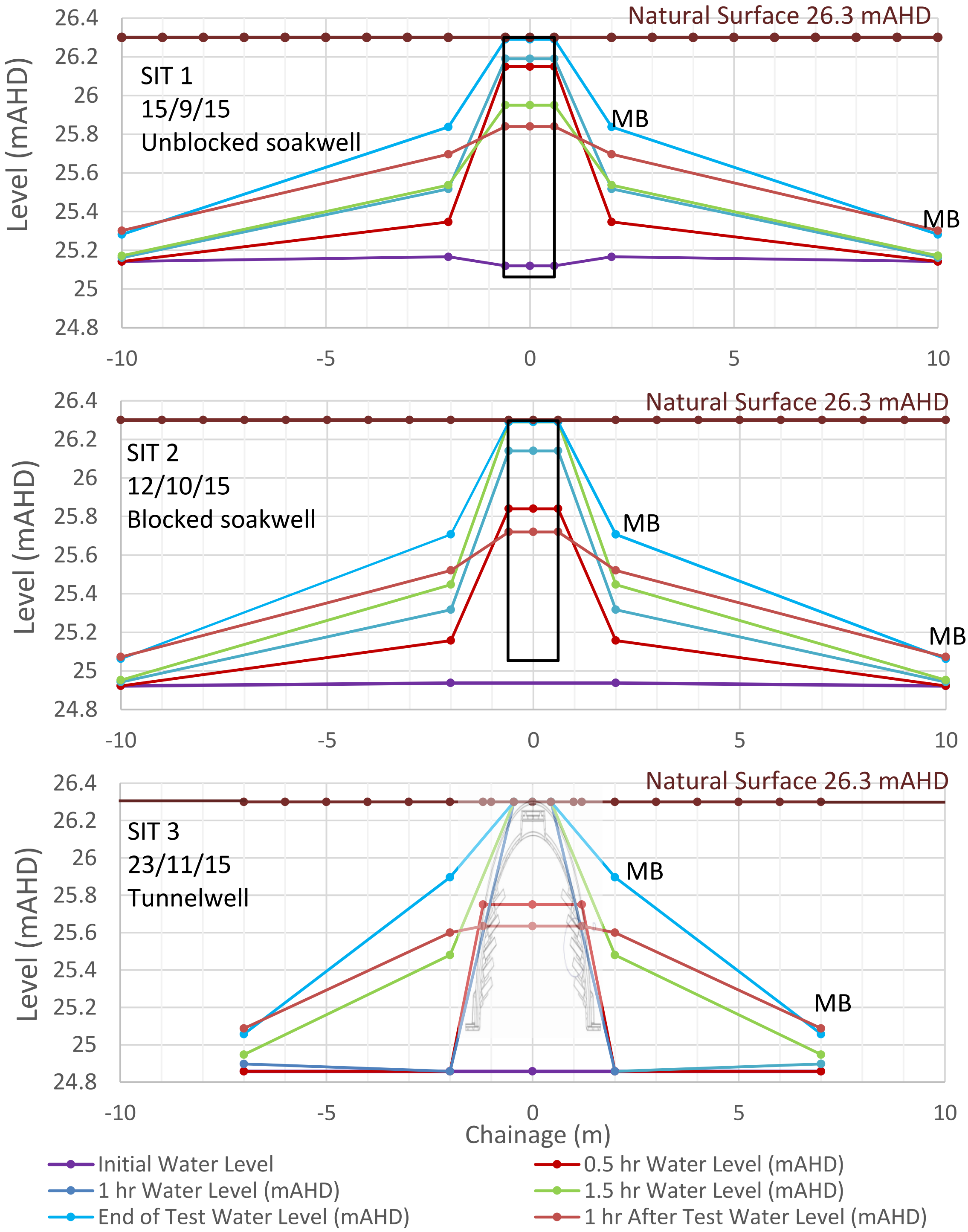

Figure 13: Cross sections of unblocked soakwell SIT 1 (top), blocked soakwell SIT 2 (middle) and Tunnelwell SIT 3 (bottom). MB = monitor bore.

4.5 Effect of holes in Soakwell and Tunnelwell

4.5.1 Soakwells

The soakwell has holes both in the sidewall (slots) and base as follows:

(i) Sidewall slots
Number = 18
Size (L × H) = 145 mm × 45 mm
Each slot area = 0.0065 m^2
Total sidewall slot area = 0.12 m^2
Total sidewall area = 4.52 m^2
% slots of total sidewall area = 0.12/4.52 = 2.6%

(ii) Base hole area
Number = 1
Diameter = 0.2 m
Area = 0.031 m^2
Base area = 1.13 m^2
% holes of base area = 0.03/1.13 = 2.8%

(iii) Sidewall slots + basehole
Total open area = 0.12 + 0.031 = 0.15 m^2

4.5.2 Tunnelwell

The Tunnelwell has louvres in the side walls, with solid end caps. The base of the Tunnelwell is completely open with the arch laid directly onto the sand.

(i) Sidewall louvres
Number = 126
Diameter = 39 mm
Each louvre area = 0.0012 m^2
Total sidewall louvres area = 0.15 m^2

(ii) Base open area
Size (L × H) = 1165 mm × 1312.5 mm
Base open area = 1.53 m^2
% hole of base area = 1.53/1.53 =100%

(iii) Sidewall louvres and base hole area
Total open area = 0.15 + 1.53 = 1.68 m^2

4.6 Discussion

As with boreholes for groundwater abstraction, the greater the open area of a slotted screen, the lower velocity of water flow through the slots, and the lower head required for water to pass from the outside to the inside of the screen. This head is called "well loss", and it is desirable to minimize it, by maximizing the screen open area.

Similarly with infiltration devices, it is desirable to maximize the open area.

The Tunnelwell has larger open area than the equivalent soakwell. The open area of both was adequate and resulted in minimal head loss between the inside and the outside of the infiltration device. However the Tunnelwell with greater open area is superior, especially if clogging is likely to occur over time.

To answer the cryptic question posed by the paper title "How many holes does one soakwell need?" – the results show that the multiple small holes and base hole in both soakwell and Tunnelwell are adequate, so that increasing the number or size of holes would not increase infiltration rate.

In summary both soakwell and Tunnelwell have sufficient open area.

It is the hydraulic conductivity and the water table which limit the infiltration rate, rather than the size or number of openings of the soakwell and Tunnelwell.

4.7 Stormwater Runoff Estimation

The infiltration results are consistent with previous testing by JDA at a housing lot scale of soakwells (JDA, 2015), which shows zero runoff occurred in a 10 year ARI 12 hour duration storm applied using water tanker. This report is available from the authors on request.

The infiltration testing results in this paper again indicate that lot runoff coefficients are close to zero for storm events on lots with soakwells.

Alternatively, this finding can be stated as the initial loss (IL) is equal to the infiltration device volume of 13 mm and continuing loss (CL) is 32.5 mm/hr.

The rate of continuing loss (CL) for different water table depths and soil hydraulic conductivity values can be estimated using groundwater flow equations in models such as MODFLOW and FEFLOW.

In City of Armadale (2015) fraction imperviousness is suggested as the basis for estimating runoff rates using a 10 ARI Rational Method runoff coefficient based on fraction imperviousness, see Table 5 below and Figure 14 reproduced from City of Armadale (2015). City of Armadale (2015) states that this method is intended as a robust solution which is easy to apply for small scale projects. The publication also states that the Rational Method and Table 5 is not encouraged for most sub-divisions and should be considered for simple applications only. Whilst these may be appropriate for eastern states stormwater runoff estimation, this paper argues that they are overly conservative when water sensitive urban design infiltration practices such as soakwells and Tunnelwell devices are used as is common on the coastal plain around Perth.

The infiltration testing results reported in this paper confirm that the use of impervious area as a parameter for estimating stormwater runoff rates in Perth on sandy soils may result in gross overestimation.

TABLE 5: IMPERVIOUS AREA PERCENTAGES FOR DIFFERENT LAND USES (CITY OF ARMADALE, 2015)

	C_1	C_5	C_{10}	C_{100}
Residential lots	0.56	0.67	0.70	0.84
Access streets and road reserves	0.64	0.76	0.80	0.96
Group housing sites, mixed use commercial/ residential, local centre & laneways	0.72	0.86	0.90	1.00
POS basins	0.72	0.86	0.90	1.00
POS remaining areas	0.08	0.10	0.10	0.12

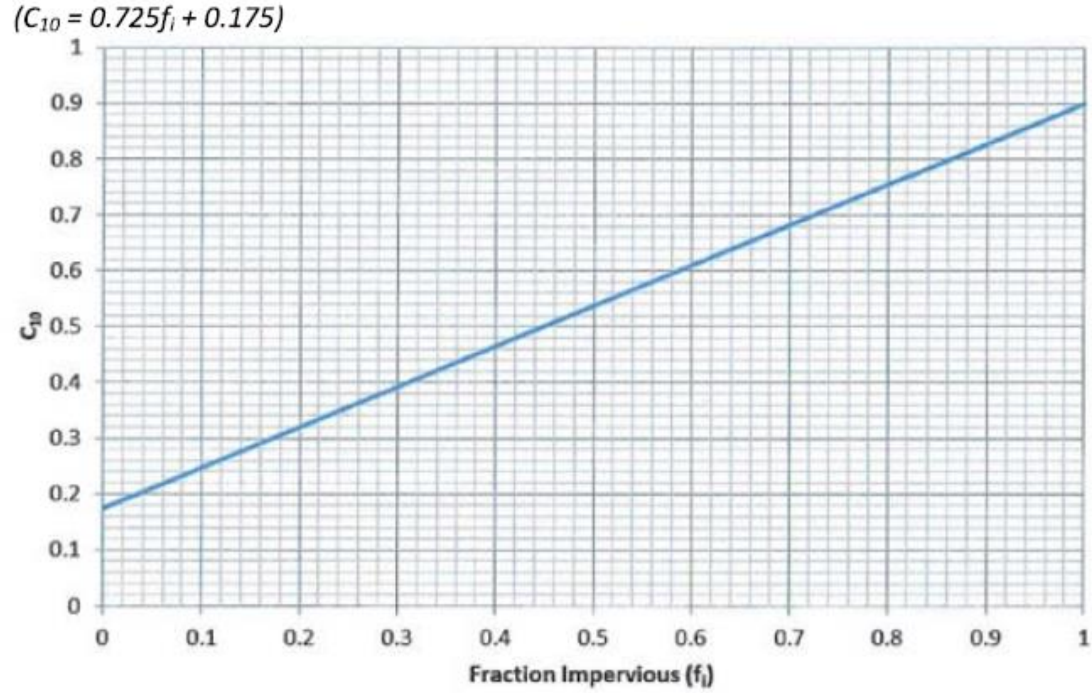

Figure 14: Calculating C_{10} from Fraction Impervious

A subsequent publication prepared for the City of Armadale (Essential Environmental, 2015) contains the following relevant paragraphs:

"The fraction impervious is useful for developing an understanding of the runoff that is likely to be generated by each land use, how this runoff behaves thereafter and how much of it enters a waterway or drainage system must be the subject of site specific analysis. Furthermore the way that this is represented as a 'runoff parameter' will depend on the modelling approach used.
For example, a mid-sized urban residential lot on clayey soils with an FI of 6% may be considered to contribute as much as 80% runoff with very small initial losses where it is provided with a direct connection and no soakwells or other on-site retention system. Conversely, if the lot is sandy, provided with soakwells and has no piped connection it may contribute far less at around 40% to account for continuous infiltration from the soakwell and have a higher initial loss to account for its volume."

The authors support the logic of these paragraphs and reiterate the need for actual data, such as reported in this paper, to be referred to when estimating runoff rates from sites with soakwells.

4.8 Summary

With a shallow water table, the soakwell and Tunnelwell infiltration devices, both with void capacity 1.4 m^3, infiltrated 15 kL (15 m^3) of water applied from a tanker in a 4 hour period.

The number and size of holes is adequate for both soakwell and Tunnelwell, and increasing the number or size of holes would not enhance infiltration rate.

Assuming designed to hold 13 mm from an impervious catchment area, the void volume of 1.4 m^3 corresponds to an impervious catchment area of 1.4 m^3 divided by 13 mm equals 105 m^2.

The difference between the applied volume (15 m^3) and the void volume (1.4 m^3) is 13.6 m^3, and this volume infiltrated in a 4 hr period corresponding to 13.6 m^3 divided by 4 hour equals 3.4 m^3/hr, or 3.4 m^3/hr divided by 105 m^2 equals 32.5 mm/hr from the impervious catchment area.

These results can be expressed as an initial catchment loss (IL) of 13 mm, followed by a continuing loss (CL) of approximately 32.5 mm/hr.

The continuing loss of 32.5 mm/hr is equal to an infiltration rate of 3.4 m^3/hr divided by 1.14 m^2 equals 3.0 m/hr, or 72 m/d, over the base area (1.14 m^2) of the soakwell. This infiltration rate of 72 m/d is six times the soil hydraulic conductivity (K) of 12 m/d.

Similarly the continuing loss of 32.5 mm/hr is equal to an infiltration rate of 3.4 m^3/hr divided by 1.94 m^2 equals 1.75 m/hr or 42 m/d over the base area of the Tunnelwell of 1.94 m^2. This infiltration rate of 42 m/d is 3.5 times the soil hydraulic conductivity (K value) of 12 m/d.

A more realistic interpretation of the infiltration rate would use the full surface area of the soakwell, including both sides and base area which total 4.52 m^2 plus 1.14 m^2 equals 5.66 m^2 (see Section 7.1).

The infiltration rate calculated using this flow area is 3.4 m^3/hr divided by 5.66 m^2 equals 0.6 m/hr or 14 m/d, which approximates the K value of 12 m/d.

An infiltration rate similar to K indicates a hydraulic gradient close to 1.0, as K is defined at unit hydraulic gradient.

In reality the hydraulic gradient in the recharge cone gradually decreases from a maximum value around the soakwell itself, to much lower values several meters away from the soakwell. At all locations the governing groundwater flow equation (Darcy's Law) is satisfied such that flow rate equals K times hydraulic gradient times flow area.

Using these results, continuing loss rates (CL) for different depths to water table and hydraulic conductivity can be calculated using groundwater flow equations in models such as MODFLOW and FEFLOW.

This data could be used by local authorities as a guide to likely runoff rates on sites with infiltration devices such as soakwells and Tunnelwell.

5 Conclusions

Underground infiltration devices (Soakwell, Tunnelwell etc) provide both initial loss and continuing loss for storm rainfall.

The experiments reported are thought to be the first of their type in WA.

The experiments suggest very low, or no runoff, for lots with sandy soil, and infiltration devices.

6 References

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Western Australia Geological Survey, Bulletin 142.

7 Acknowledgements

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