HOW OFTEN WILL SEWAGE SPILL?

[Stochastic Analysis & Design of a Waste-Water System with large RDII components] v1.3

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ABSTRACT

For systems with large RDII (Rain Derived Infiltration and Inflow), **classical** methodology for estimating loading on Waste-Water systems **can be dangerous** if it portrays the impression of a known ultimate design flow. The reality, for such systems, is that ultimate loads <u>cannot</u> be determined. The paper presents real-world **modelling to enhance planning and environmental aspects,** of a sewer-system *backbone* for a "wet" area [1500 mm/year rainfall located at the base of steep sided valleys] by predicting the stochastic nature of outflows and overflows. Described is:

1. The use of RDII analysis to **predict loading probabilities by**;
   a. Analyzing flow data to identify dry-weather loads then subtract same to determine wet-weather components. (The latter "**left-overs" proved to be the "main-course**" and represent up to 90% of the load during times of high rainfall!)
   b. Formulating a mathematical relationship between rainfall and RDII then use same to estimate loadings for various return periods.
2. Design & **prediction of overall discharge frequencies and characteristics**; by applying the load predictions, and modelling various scenarios, to ensure that, on average;
   a. For annual events; all WW is treated and zero overflows occur
   b. Every 2 yrs, excess flows receive simple treatment in a bypass system (but are still discharged through the normal diffusers into tidal flushed receiving waters)
   c. Every 5 yrs, some of the bypass treated effluent is discharge to flooded turbulent streams
   d. Every 10yrs, some untreated effluent is overflowed but only to flooded turbulent streams.

KEYWORDS

Sewage, Sewer, Waste Water, Stochastic, Return period, ARI, Overflows, Modelling, SWMM, RDII, RDIIA, Reality, Inflow, Infiltration, NZ4404

CONTENTS

1 INTRODUCTION

1.1 PREAMBLE

Hydraulic modeling tools are becoming more useful but, the *threatening cloud* of climate-change[a] and the desire for *No Overflows*, necessitates some reality checks. By definition, all models are wrong – some are just less wrong than others, because….

Modelling can <u>only</u> be an approximation of reality -- especially future loading.

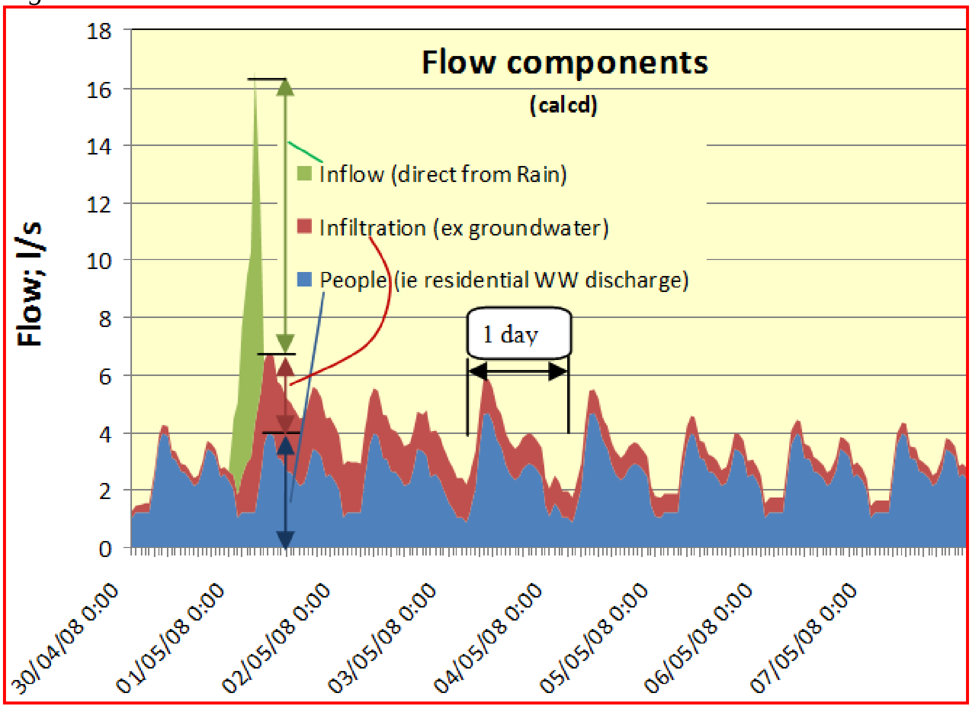
The stochastic nature of model loading and the consequences thereof, are the main foci of this paper.

For systems with large RDII (Rain Derived Infiltration and Inflow), classical methodology for estimating loads often portrays the false impression of a known ultimate design flow. The reality, for such systems, is that ultimate loads <u>cannot</u> be determined. The paper presents a case study of some real-world modelling of a sewer-system *backbone* for a "wet" area [1500 mm/year rainfall located at the base of steep sided valleys] by predicting the stochastic nature of outflows and overflows.

Earlier research [Reference (3)] and data analysis for a similar catchment in the same region showed RDII to be dominant. Figure 1 shows a typical time sequence during a rain event while figure 2 shows the relative components of the peak flow.

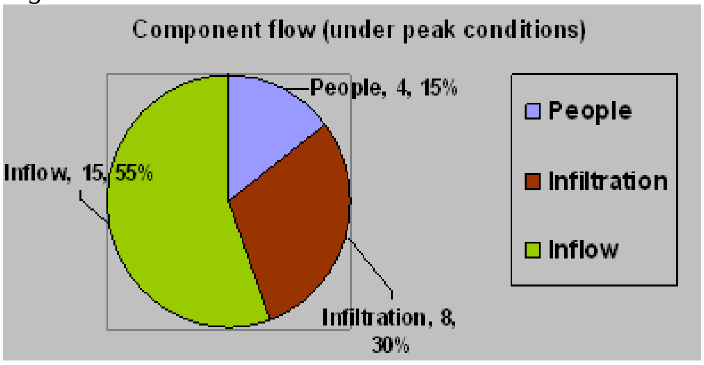
Only 15% of the load comes from WW discharges!

Figure 1



[a] From Ref(6) "Wastewater systems are very sensitive to changes in rainfall patterns and the cost implications of climate change are large". "Growth determines when upgrades occur, the level of service gives the size the asset needs to be, any climate
change effect is a safety factor additional to the LOS."

Figure 2



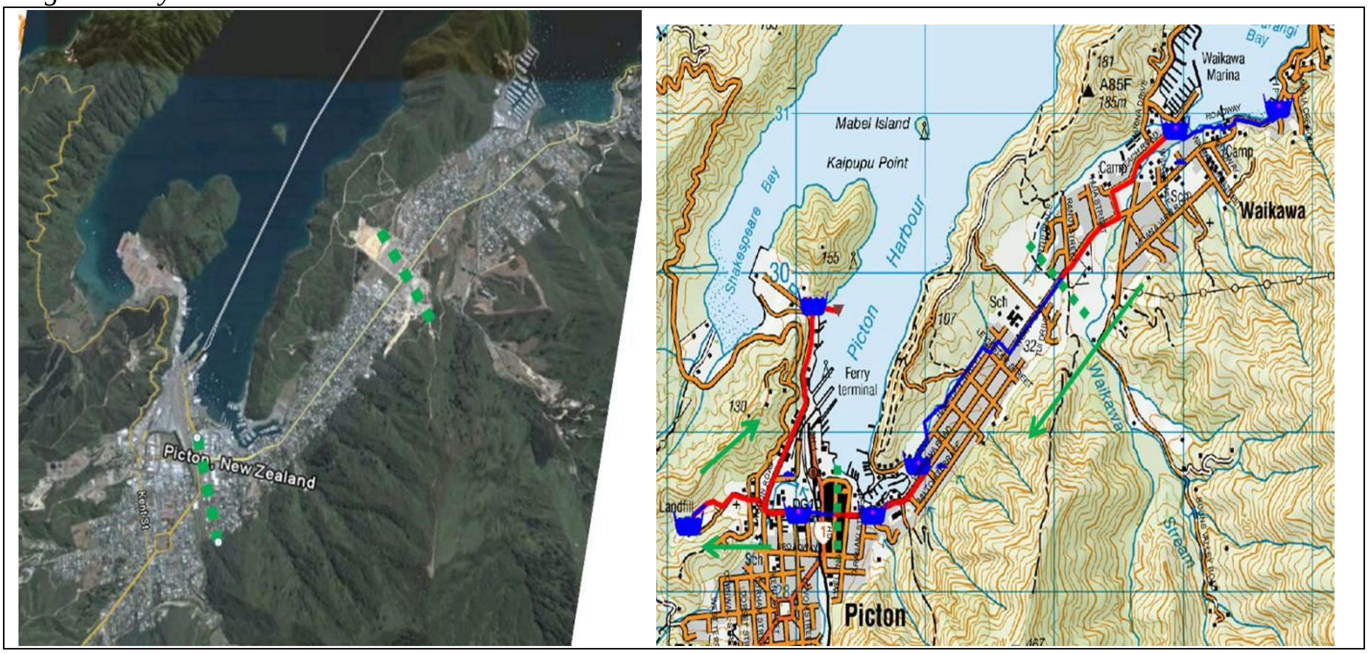
This case study looks more closely at the RDII components on a stochastic basis, and then uses the results for design studies. The followings steps are covered;

1. **Analyse** recorded data to identify components – especially RDII
2. **Determine** load v ARI (ie Return Period)
3. **Compare with classical** approaches
4. Use results to **refine the preliminary design** and
5. Examine, & ***deal with* Overflow** situations & finally;
6. **Summarise & make observations**

1.2 LAYOUT

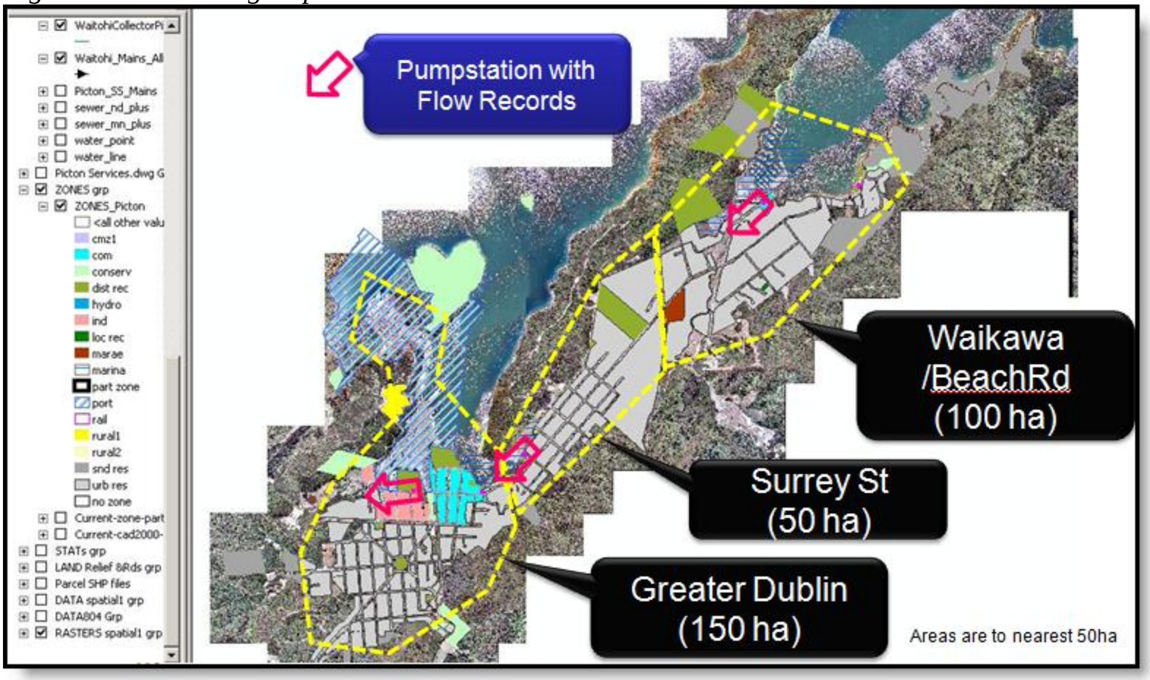
The figure below shows the WW system in question (Picton - Waikawa NZ).

Figure 3 Layout



The catchments are shown below.

Figure 4 Catchment groups and zones

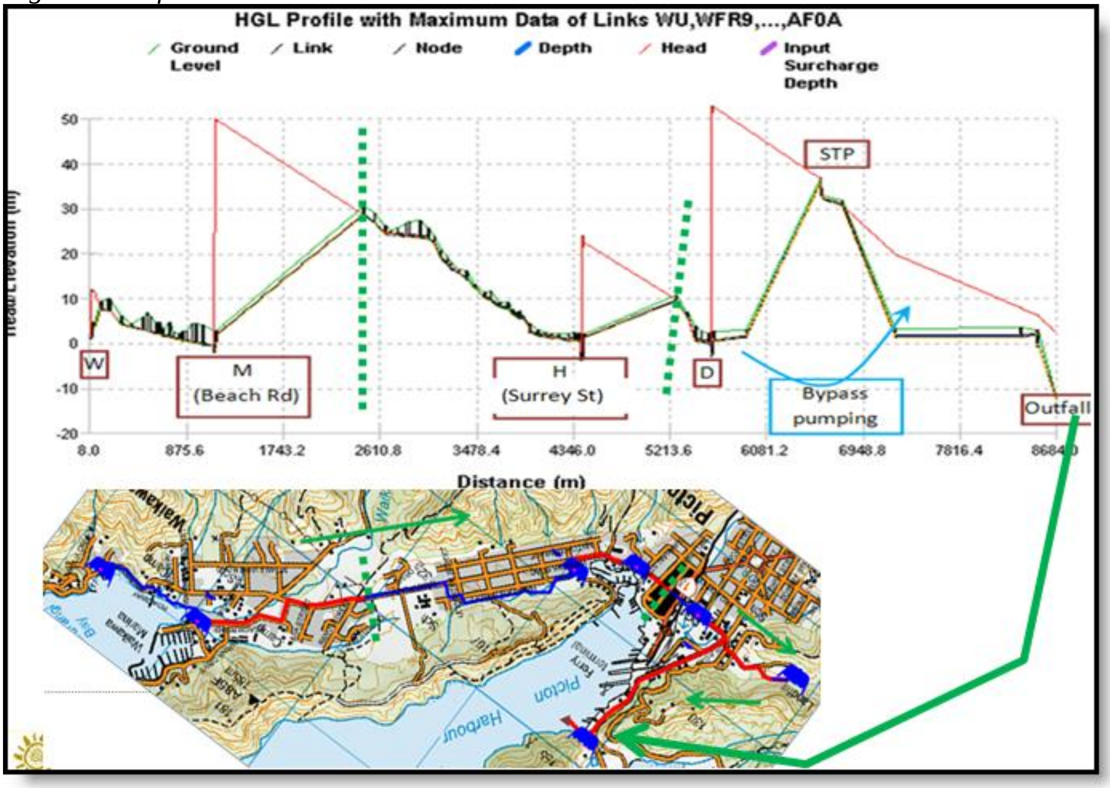


Note the steep sided narrow nature of the Surrey St catchment highlighted in yellow in the figure below.

Figure 5 3D view

The figure below shows the profile along the backbone. Two pumpstations (M & H) lift the WW to ridge lines (green dotted) for gravity flow to the next pumpstation and finally at pumpstation, D, pumps to the STP. A gravity outfall pipe flows to the sea outfall and the bypass system will handle [some] loadings in excess of STP capacity.

Figure 6 Profile



The existing population is around 7000 and the area 300ha. Allowance for growth is approximately 100% (for both metrics).

1.3 SITUATION

At present, along the backbone, there are: old pipes, failures, overloading, overflows & lack of spare capacity. So…

The entire backbone needs to be replaced.

The question is: what should the design capacity be and what are the overflow consequences?

Note for the reader: Considerable detail is included for steps 1 to 3 to provide reference material for reviewers and to assist others who may want to utilize the methodology. If the reader needs only to consider the effects of using stochastic loading, then skip to section *2.7 Conclusion from the Analysis* .

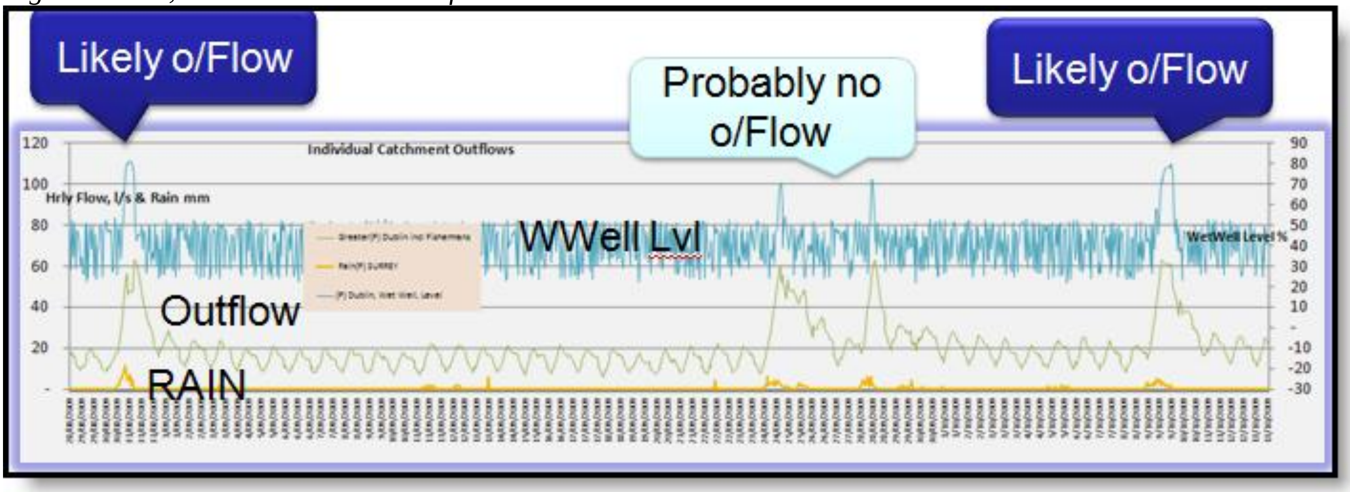
2 CASE STUDY; RDII ANALYSIS

2.1 DATA

A typical data sequence is shown below. The large effect of the RAIN upon Level and Outflow can be clearly seen.

Even a mild rain event trebles the outflow!

Figure 7 Rain, WetWell level and Outflow



NB The only availabe data is PS outflow NOT catchment load (which is what we want) or even WetWell inflow (which is what we *really really want*!).

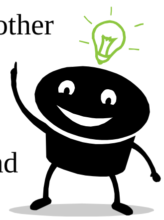
The data is not "clean" *[it never is]* so it was necessary to use; local knowledge, field staff experience, and visual interpretation to identify clean sections with high flows, but dodge; overflows (which were not recorded) and bad or missing data.

2.2 DWF

Industry standard data analysis tools[b] as recommended by EPA in a 2008 study were used to identify dry weather sequences and extract the typical diurnal flows.

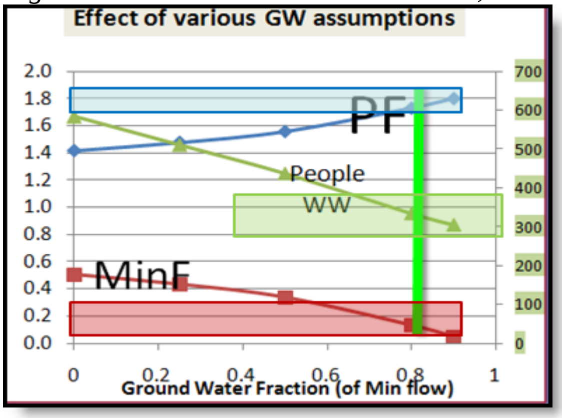
Spreadsheet calculations were then used to find sensible combination of discharge per capita (People WW), peaking factor (PF) & minimum factor (MinF), and groundwater infiltration fraction (GWIFraction). This is illustrated in the figure below.

Note: the answers had to be believable (in relation to; experience & comparisons with other areas) but absolute accuracy is not vital -- as DWF is overshadowed by RDII. However, the technique was found particularly valuable as a reality check because, if a believable combination was not found [eg the GREEN vertical line] then data needed to be examined further. In the figure below; the shaded boxes represent believable ranges for each curve, and the green line illustrates the chosen combination of: *PF*=1.7, *MinF*=0.1, *People WW*=340 l/p/d and a GWIFraction of 0.8. (A Lower *GWIFraction* would mean less believable peaking, and a *People WW* too high.)



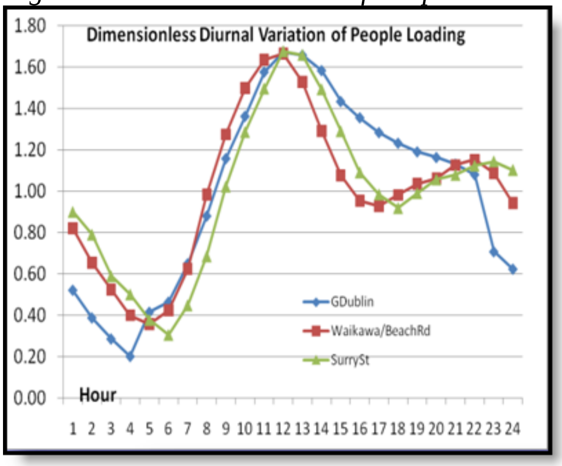
[b] infoSWMM RDIIA. Theory and more details are in Appendix>App1.

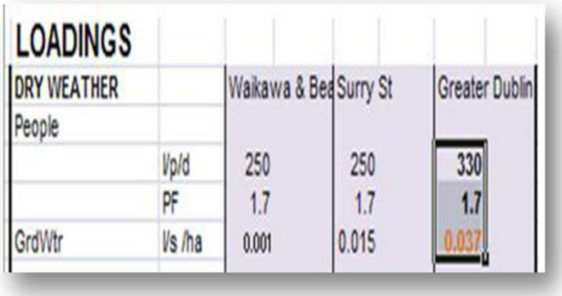
Figure 8 DWF. LHS scale=PF & MinF, RHS= PeopleWW l/p/d



The resulting dirurnal variation curves are shown below. The pattern if very similar to that obtained by analysing water demand. The different shape for the Greater Dublin St catchment *makes sense* because the catchment has a siginificant commercial/industrial component.

Figure 9 Dirurnal Variation of People DWF loading



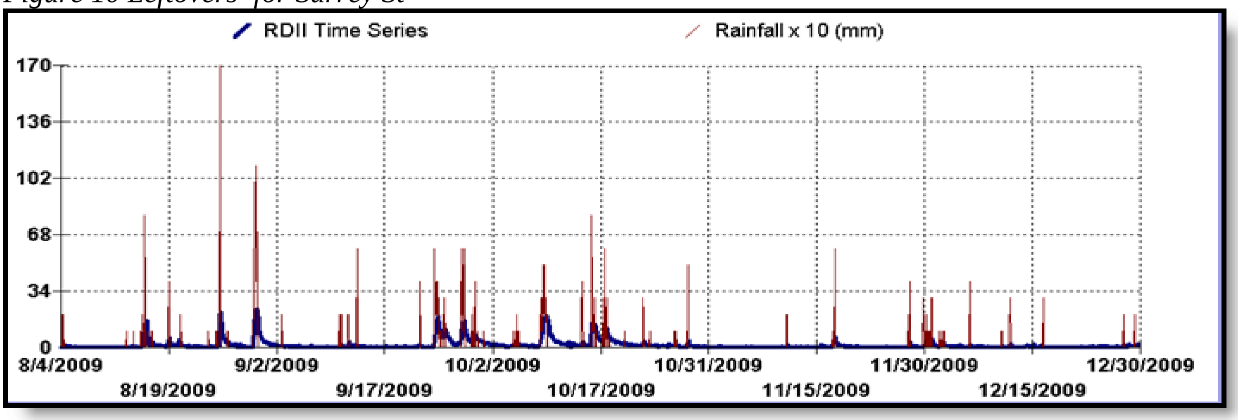


LOADINGS				
DRY WEATHER		Waikawa & Bea	Surry St	Greater Dublin
People				
	l/p/d	250	250	330
	PF	1.7	1.7	1.7
GrdWtr	l/s /ha	0.001	0.015	0.037

2.3 WW COMPONENTS

Subtraction the DWF from the records gives the Wet Weather components, i.e;   the *leftovers* shown below.

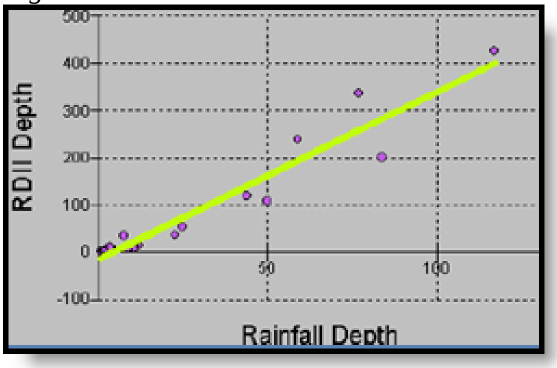
Figure 10 Leftovers[c] for Surrey St



The correlation between Rainfall and RDII is quite clear as illustrated below

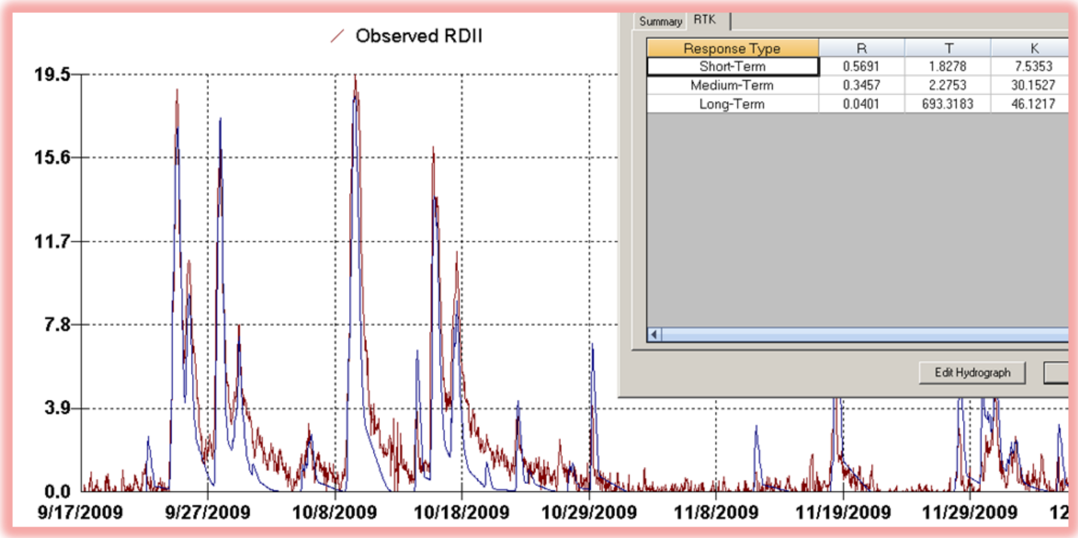
[c] After subtracting DWF

Figure 11 Correlation



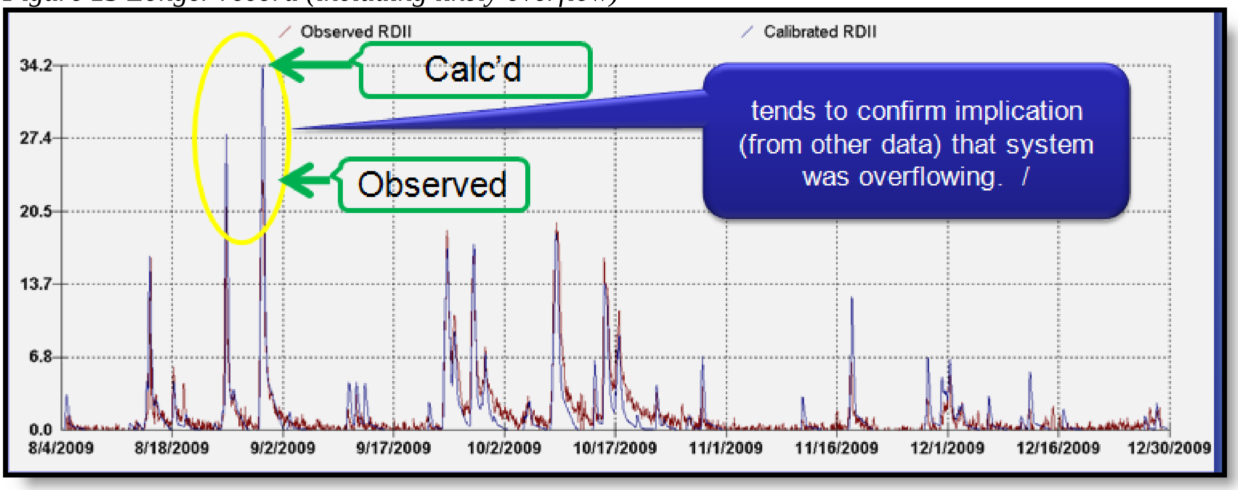
Using a non-overflowing period gave SUH results shown below. Peaks correlate well… recession not so well; but it is the peaks that are of much greater importance in this case, so the correlation is deemed sufficient.

Figure 12 Short record; Calculated v Observed RDII



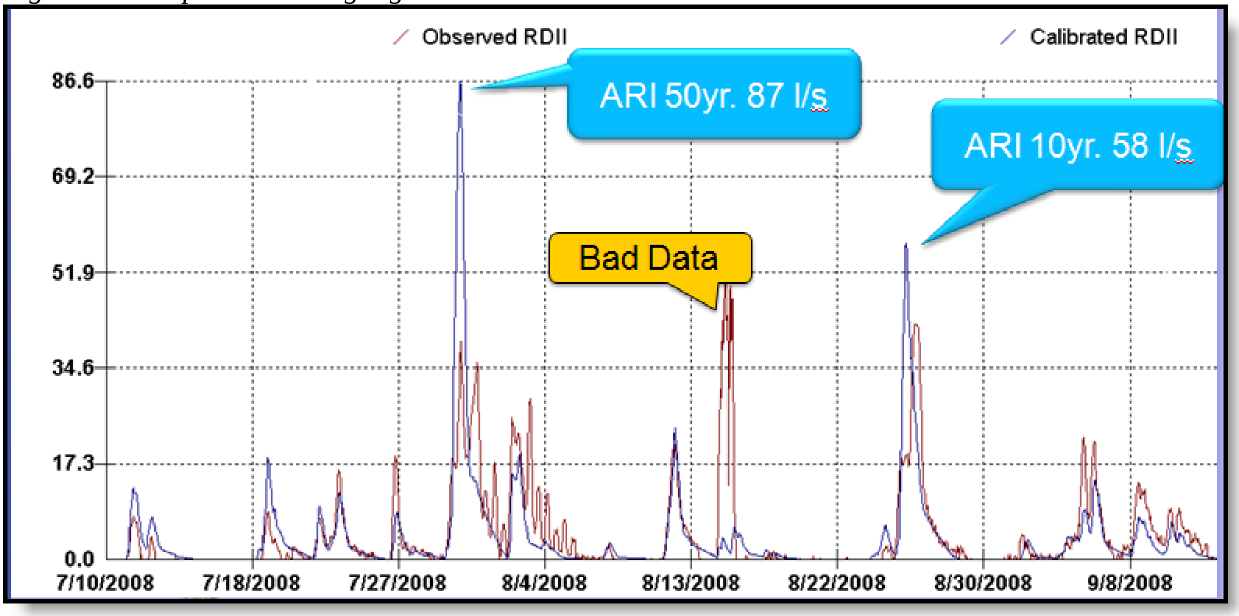
Applying this UH to a longer record gives the comparison below. This tends to confirm the existance of an overflow.

Figure 13 Longer record (including likely overflow)



Applying the SUH again to encompass events of larger ARI yeilded the results below.

Figure 14 Comparison during higher ARI rain events



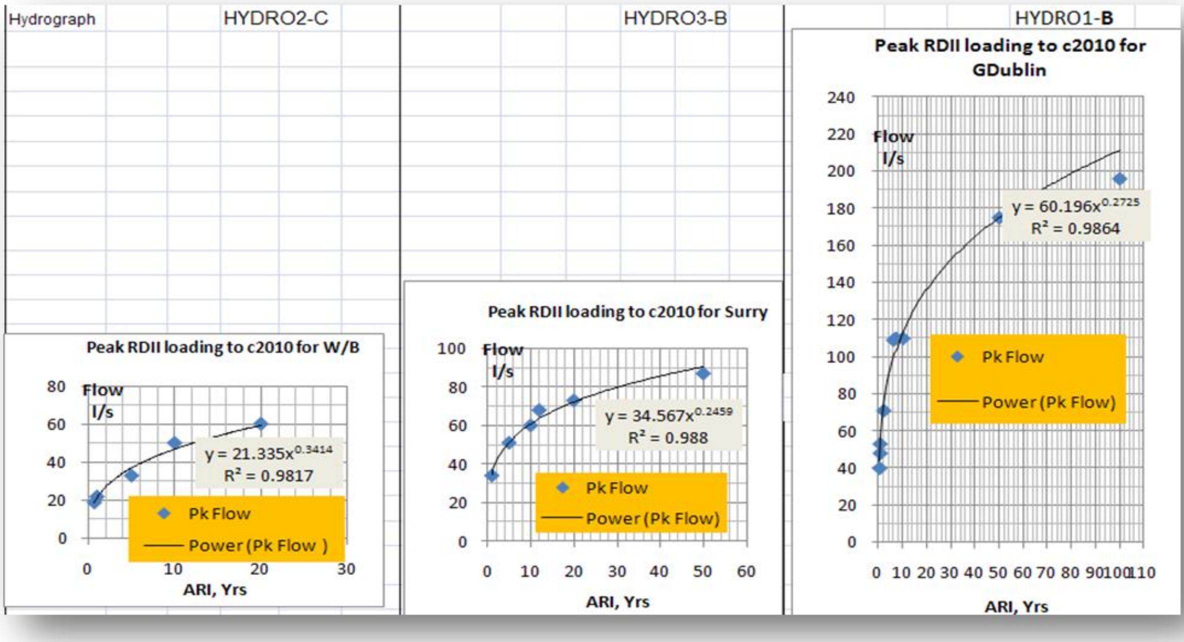
Notes:

1. For all of this analyis, ARIs are based on historical events and make no allowance for climate change. Indicatively they could halve with climate change [See Ref(8) & Appendix at ***6.2***]. A 20 yr event could become a 10yr event!
2. Actual rain events were used plus 1 synthetic to fill-in low ARI gap
3. Analysis showed, that **longer events** (lower intensity) generally provided **larger RDII** than shorter events of same ARI:
   a. Typically 12-24 hr events gave a greater RDII than 3-6 hr events.
   b. This is similar to situation found elsewhere in MDC similar to findings by others[d].

[d] Eg Verbal comment by presenter of Ref(7)

Plotting all the results yielded the following ARI v Peak flow curves. All relationships are similar (*which is good*).

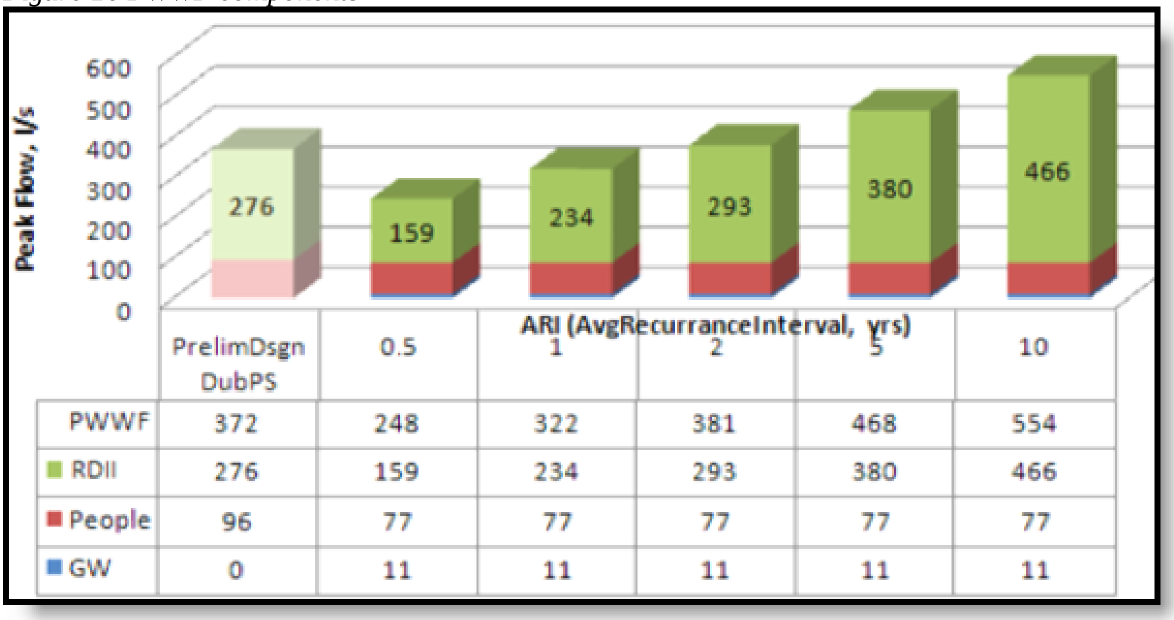
Figure 15 Peak Flow v ARI



2.4 PWWF

Assembling the entire *picture* gives the following chart. The *PrelimDsgn* block at the start was based on a fixed ultimate loading assumption. It thus represents less than a 2 yr ARI.

Figure 16 PWWF components



	PrelimDsgn DubPS	0.5	1	2	5	10
PWWF	372	248	322	381	468	554
RDII	276	159	234	293	380	466
People	96	77	77	77	77	77
GW	0	11	11	11	11	11

A classical ultimate-design-loading case appears to fall short and represent less than a 2 yr ARI (and with Climate change, perhaps only a 1 yr ARI)!

2.4.1 LOADING FROM OUTSIDE THE CATCHMENT

Two catchments exhibited greater flows than were believable; given the amount of rain on the catchment:

For the Dublin Catchment; to keep the R term reasonable (and totalR <1 [e]) it was necessary to use a catchment area greater than that covered by the sewer infrastructure. It is considered likely (from anecdotal and field experience) that the large stream through the catchment is bringing water from outside the catchment, some of which enters the sewer system -- and this adds to the RDII. This would explain the larger area value.

For the Waikawa/Beach catchment; some RDII spikes correlated better with rain records further U/S & outside the catchment. Again; an indication of RDII from the stream flowing through the catchment.

2.5 RESULTS

To provide a comparison and to provide simple metrics, the results were converted to l/s /ha. Assumptions adopted for future design considerations were as follows;

> For growth allowance; first the existing area was in-filled, then growth areas were added. Infill assumptions were (from field info): 70% of I&I assigned to mains (ie existing) and 30% to service connections. The latter was thus applied to in-fill areas as an extra load (because for infill areas, the mains exist but not the connections)
>
> A minimum of 0.4 l/s /ha for 2 Yr ARI was imposed (to align with information & loads used by others in similar NZ regions).
>
> Growth areas were assumed to have I&I rates = 70% of that for existing. [Comment: Analysis of recently installed systems has shown disturbing I&I only a decade or so after construction. A figure of 70% may still be a little high but give the nature of the topography, groundwater and conditions in the area (further discussed below) a lower number is deemed to be too risky.]

Sensitivity analysis was also carried out to refine assumptions & check consequences. The above set is the final group chosen. The overall effect was not highly sensitive to these assumptions.

The final numbers are shown in the figure below.

Figure 17 RDII Rates

RDII l/s /ha v ARI Yrs	0.5	1	2	5	10
Waikawa/BeachRdExisting c2010	0.13	0.19	0.25	0.34	0.42
Waikawa/BeachRdExisting areas	0.16	0.24	0.31	0.43	0.54
Waikawa/BeachRdJust Growth Area	0.11	0.17	0.22	0.30	0.38
Waikawa/BeachRd ULTIMATE	**0.14**	**0.20**	**0.26**	**0.36**	**0.45**
Waikawa/BeachRd Min ULTIMATE	**0.21**	**0.30**	**0.40**	**0.55**	**0.69**
SurryStExisting c2010	0.37	0.59	0.74	0.93	1.11
SurryStExisting areas	0.43	0.69	0.87	1.08	1.30
SurryStJust Growth Area	0.30	0.48	0.61	0.76	0.91
SurrySt ULTIMATE	**0.37**	**0.59**	**0.74**	**0.92**	**1.11**
SurrySt Min ULTIMATE	**0.37**	**0.59**	**0.74**	**0.92**	**1.11**
Greater DublinExisting c2010	0.30	0.42	0.51	0.63	0.77
Greater DublinExisting areas	0.34	0.48	0.57	0.72	0.87
Greater DublinJust Growth Area	0.16	0.23	0.27	0.34	0.42
Greater Dublin ULTIMATE	**0.26**	**0.37**	**0.44**	**0.55**	**0.67**
Greater Dublin Min ULTIMATE	**0.26**	**0.37**	**0.44**	**0.55**	**0.67**

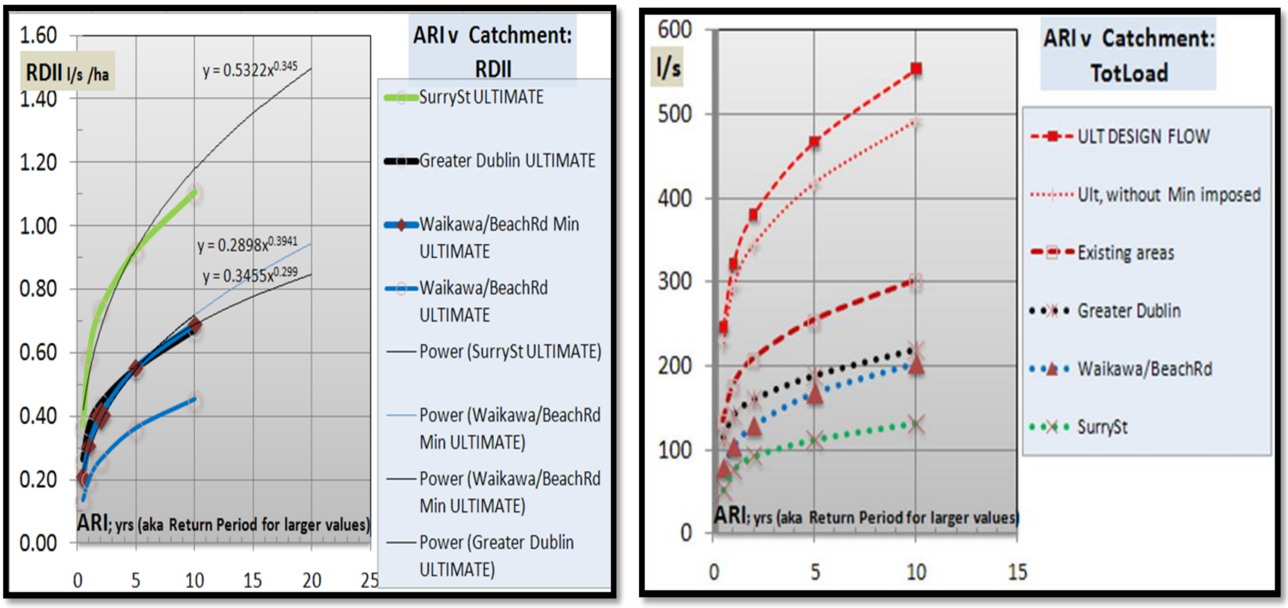
> Table notes: *Existing areas*, refers to the existing catchment area *fully infilled*. The metrics for each row are averages to that situation thus do not, and should not, add to the bottom line. The upper Waikawa/BeachRd

[e] See Appendix>App1 for theory

ULTIMATE refers the calculated values, while the 2nd row of the same name is with the *imposed minimum* applied as discussed above.

RDII rates and the resulting total loads are shown below

Figure 18 RDII Rates & Total Load Respc



The following observations are made;

1. The Surrey catchment has the highest I&I -- which is quite believable given the steep valley sides and general shape of the topography (as illustrated earlier in ***Figure 5 3D view***). Field visits have observed overland and subsurface runoff down the valley sides at the same time as high inflow is observed through joints in manhole liners and laterals. During maintenance and repairs groundwater is often observed flowing down the line of the pipe in the granular backfill (ie "*searching for the next gap / break in the system to enter the pipework*")

2. The other two catchments are similar to one another.

3. Dublin St has highest dry weather groundwater infiltration [as expected]

4. An empirical trend-line exponent between 0.25 and 0.35 seems typical; ie RDII (l/s /ha) = CC x ARI^{p} where: CC=is a Catchment II Coeff, & p is a time (ARI) dependent exponent.

5. The imposed MIN for Waikawa/Beach of 0.4 l/s/ha increases total load to the STP by about 10%. (top two curves of the RHS set in Figure 18 RDII Rates & Total Load Respc)

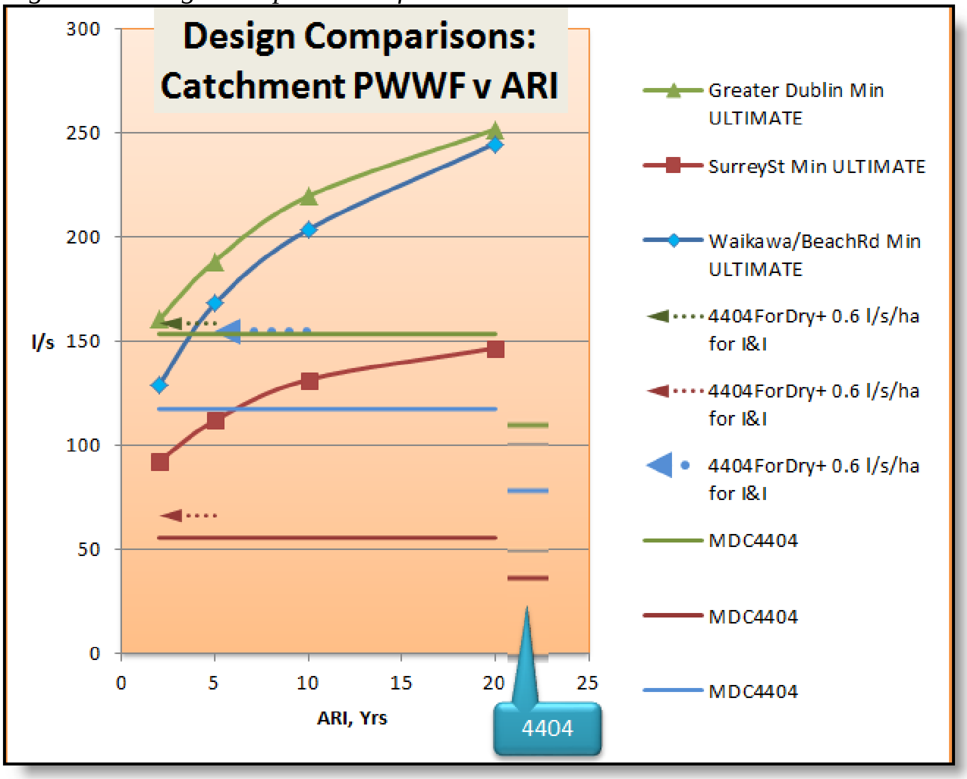
2.6 COMPARISONS WITH CLASSICAL METRICS

The figure below shows, as curves, the RDIIA results compared with classical design metrics (horizontal lines) as typified by *NZS4404:2010 Land Development and Subdivision Infrastructure*. The latter would call for a dry weather (DW) flow based on 180-250 l/p/d times a peaking factor of 2.5 then for the extra wet weather (WW) component:

- ❑ NZS4404 adds an extra 100% (ie PWWF = 2 x PDWF). [MDC NZS4404 addendum uses 200%], or

- ❑ Local practice often adds, instead, a specific extra I&I, eg 0.6 l/s/ha applied to 75% of gross Ha

Figure 19 Design Comparisons of total catchment loads



The comparison above shows that; even the MDC modified NZS4404 (horizontal lines) (which is higher than raw 4404 overlay *scrap*[f]) is still only at or below the 1 yr ARI…. and much lower than the load that is likely to occur for a 10 Yr ARI[g]. Using the fixed I&I allowance for the wet weather component instead of a simple multiplier gives the arrowed lines. Higher, but still only around a 2 yr ARI [+- for wet/dry catchments.]

2.7 CONCLUSION FROM THE ANALYSIS

The analysis leads to the following conclusion and targets;

For areas of high rainfall, and topography that directs storm water onto the catchment, classical design parameters are likely to produce frequent and unexpected sewage overflows. Maybe yearly!

- ❑ **NZS4404** (even the MDC modified local version) **loading is likely to result in considerable under-design (ie frequent overflows),** for areas like Picton.

[f] Both using the higher 250 litres per person per day. And for the Dublin St catchment 0.4 l/s/ha commercial/industrial additional I&I over 20% of the area,

[g] Which may only be a 5 yr ARI after climate-change.

- **Loads which were used for the Preliminary Design figure** (much higher than NZ4404) are still only adequate for less than a **2 Yr ARI (**see Figure 16 PWWF components **).** For this major new backbone / trunk-main system this is not deemed adequate.

- Given the stochastic nature of the loads [very large rain derived components]-- then **overflow systems will need to be incorporated into the design** eg; up to API 10yr events, convey all overflows to a large open flooded water body in a controlled manner.

- Surrey St pumpstation will need special design (because the environment is more sensitive), e.g.; a special outfall system, or all WW conveyed to Dublin St so overflow can be handled more effectively.

Also;

given the high RDII showing in the historic records, then **the local version of NZ4404 should be modified** so (1) at least sealed manhole joints and sealed lateral connections are mandatory and (2) designs must provide for higher PWWFs on a stochastic basis.

Design needs to be stochastic & incorporate high I&I values. Overflows (by design) will thus need to be planned for -- so they occur only when & where the impact is acceptable …. and can be managed.

3 CASE STUDY; DESIGN

3.1 METHODOLOGY

The design loads from the Analysis section are shown below;

Figure 20 Loading metrics

LOADINGS				
DRY WEATHER		Waikawa & Bea	Surry St	Greater Dublin
People				
	l/p/d	250	250	**330**
	PF	1.66	1.67	**1.66**
GrdWtr	l/s /ha	0.001	0.015	0.037

ARI \ Catchment		Waikawa/Beac	SurrySt Min (	Greater Dublin
2		0.40	0.74	0.44
5		0.55	0.92	0.55
10		0.69	1.11	0.67
20 Trend		0.95	1.50	0.85
20Yr Visual		*0.85*	*1.25*	*0.80*

The higher *People* loading for Greater Dublin is just the effect of counting <u>all</u> such loading, including that from commercial & industrial zones, against the Usual Resident population. The variations in dry weather Groundwater infiltration values are consistent with the comparative nature of the areas.

The above metrics were applied at each node along the backbone as follow:

PDWF= Population (serviced by the node) x l/p/d x PF

RDII = Area (serviced by the node) x l/s/ha for the ARI and catchment being considered

PWWF = PDWF + RDII

3.2 TARGET

The STP capacity is 135 l/s. This corresponds to an ARI of 1 to 2 yrs for the existing population and urban boundaries, and more frequently as urban development occurs. Beyond that, the steps investigated are described below. The specific ARI target figures for the ultimate design loads are shown in square brackets.

1. Pump all flows to the STP until the 135 l/s capacity is reached.. [<u>Output</u> from the STP will be 140 to 145 l/s because of the extra storm-water picked up at the plant itself.]

2. Treat[h] an extra 255 l/s at the bypass site then pump direct to the outfall line (thus 400 l/s at the outfall) [2 yr]

3. Treat a total of 335 l/s at the bypass site but **overflow** the extra (80 l/s) into the [flooded] stream at Dublin St. NO other overflows. [5yr].

4. **Overflow** extra extra at Dublin St and at Beach Rd <u>only</u>. [10yr]

Basically this means;

Treat & deliver all 2yr flows to the outfall;
Still treat all 5Yr flows but overflow the extra at Dublin St; & For 10Yr events, overflow the extra extra at Dublin St and Beach Rd <u>only</u>. Investigate the 20Yr ARI situation to determine what is practical.

3.3 10 YR ARI

The hydraulic profile along the backbone, up to the STP, for the 10yr ARI ultimate loading is as follows.

Figure 21 10 ARI HGLS

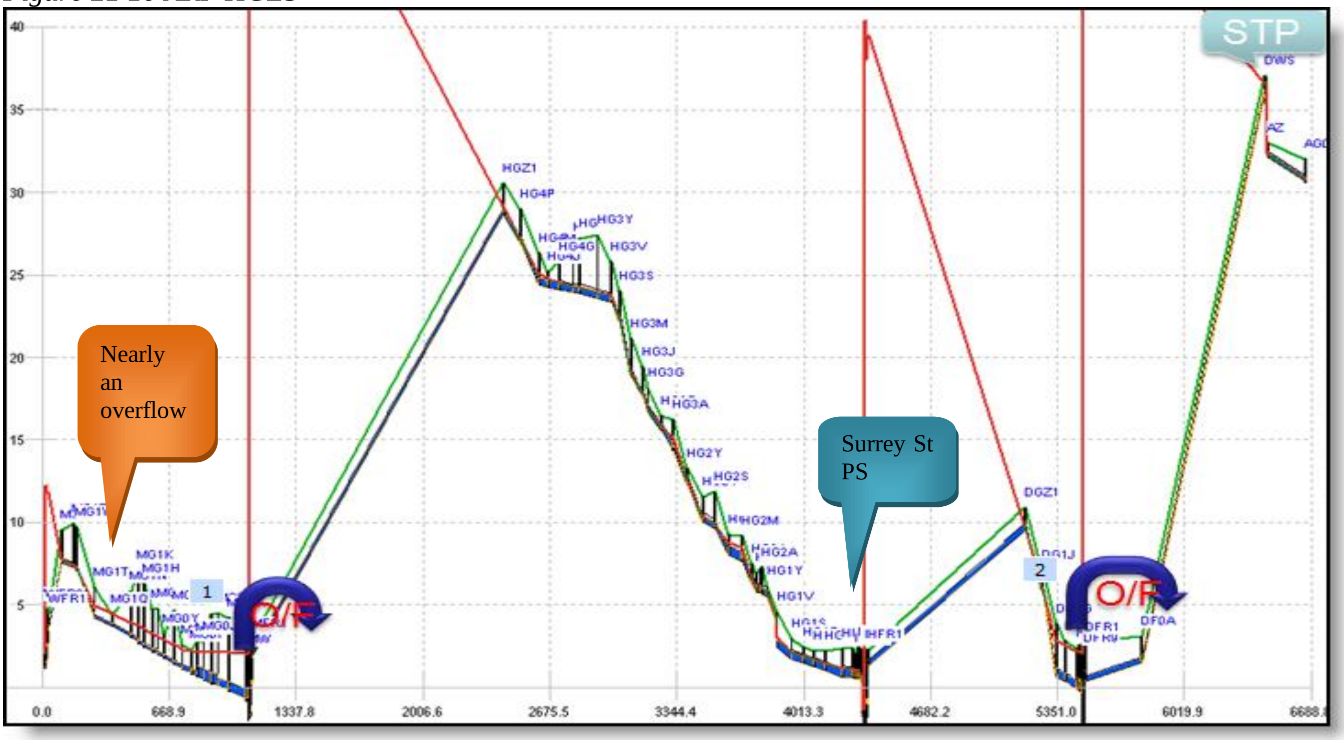


Figure Key: Red lines = HGL, Blue fill = WW (surface). O/F = Overflow

Figure observations:

1. Overflow at Beach Rd (1 LHS) and Dublins St (2 RHS) PSs.

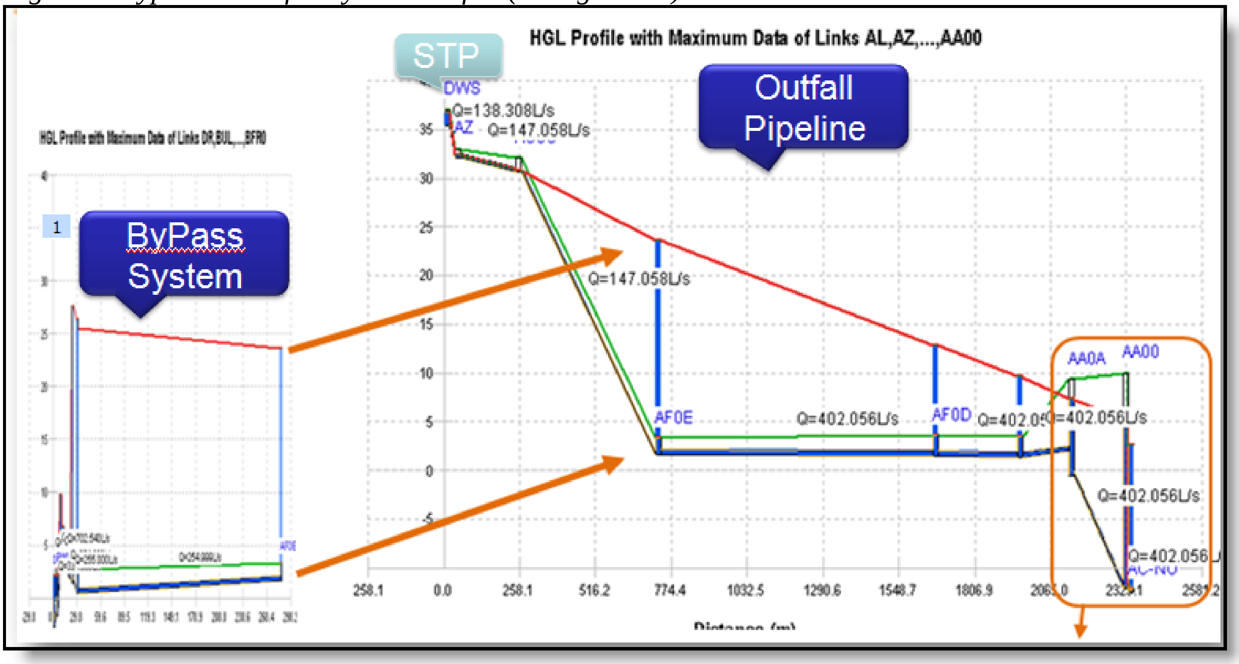
2. Full pipe for most of the long gravity section down into Surrey St PS)

3. **Nearly overflowing** upstream of the Waikawa PS

h Basic screening plus UV

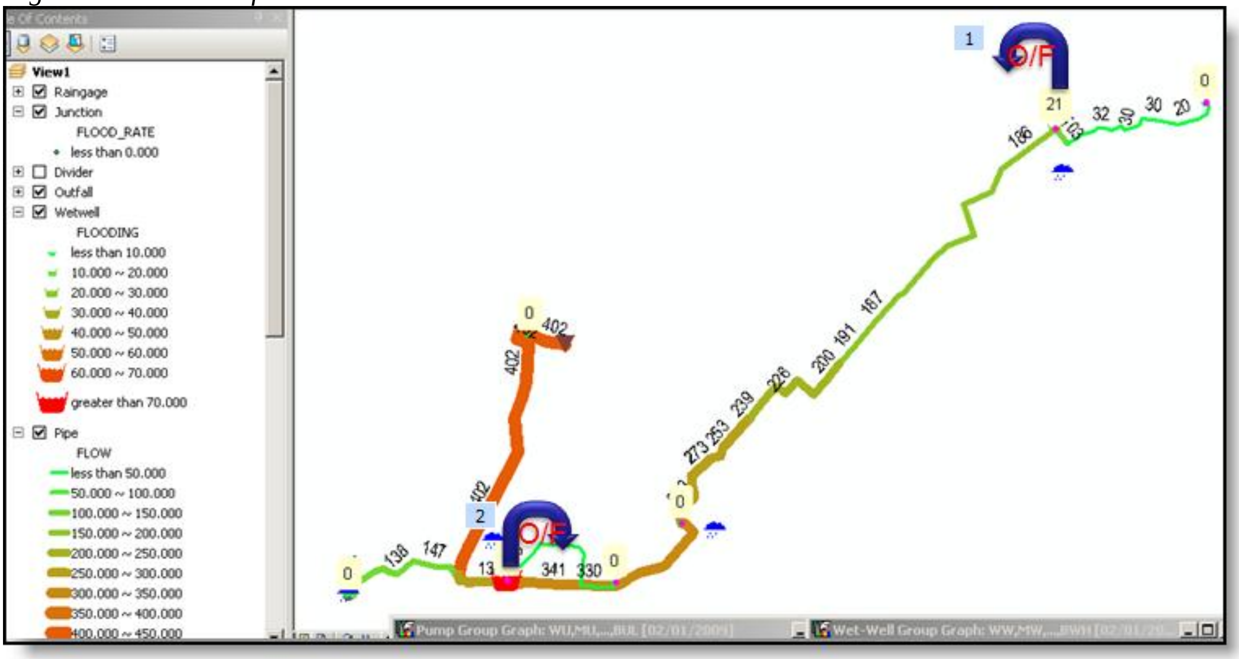
The bypass and outfall system is shown below;

Figure 22Bypass & Outfall system Profile (at High Tide[i])



The outfall pipe is full (as expected). The plan view of the entire backbone is shown below.

Figure 23 Backbone flows



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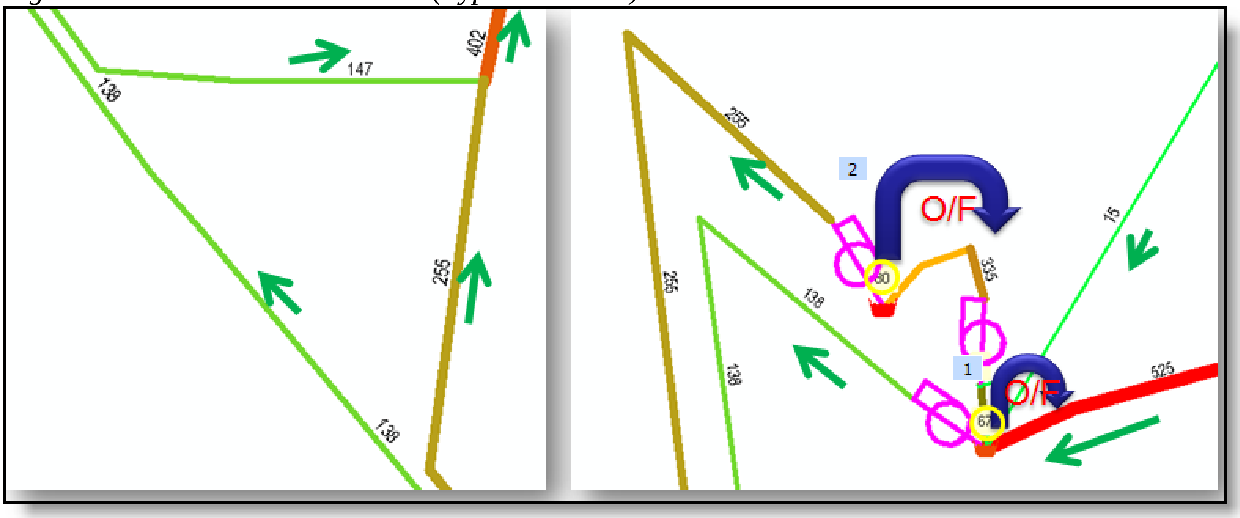
i MHWS + 1m to allow for sea level rise etc

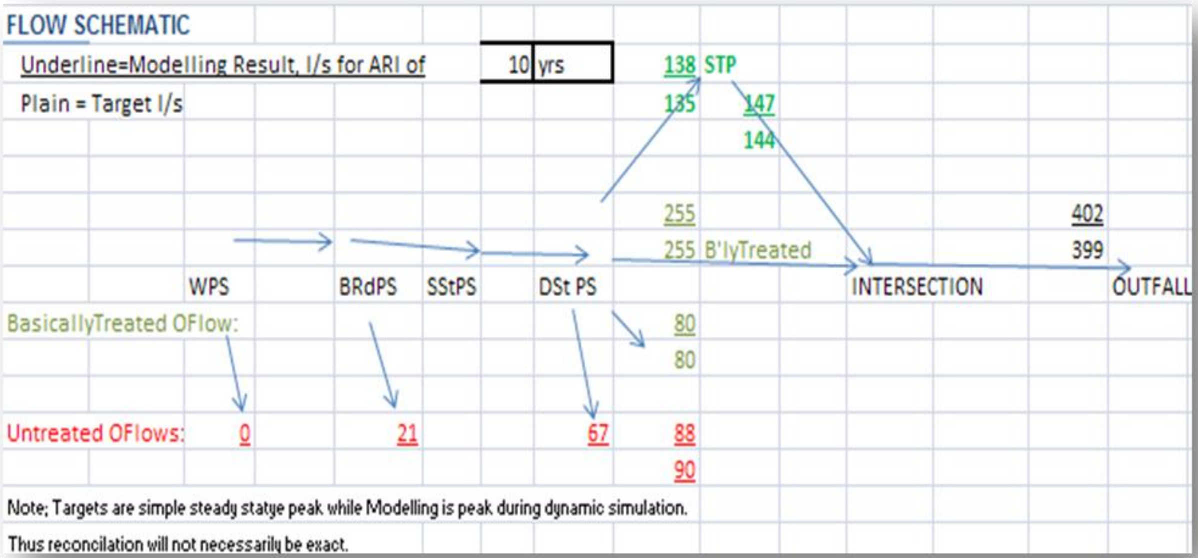
The figure below show flow details at Dublin St where there is a mainline pump to the STP plus the bypass treatment and pumping system. The LHS pane shows 138l/s being pumped up to the STP, 147 l/s coming down, and 255 l/s being pumping by the bypass system directly to the outfall line. The RHS pane shows; 1 67l/s (yellow circle) O/F from the main Wetwell, 335 l/s being given basic treatment before 255 l/s is pumped to the outfall line and the 80 l/s balance overflows 2

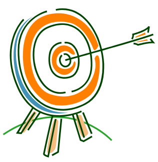
Figure 24 Details in Dublin St area (Bypass location)



The overall flow schematic is shown below and illustrates that the target regime is being met.

Figure 25 10 Yr flow schematic



Basically: 

- All 2yr ARI flows to outfall; (excess above STP capacity is given basic treatment and then sent direct to outfall line by the Bypass pumps)
- All 5Yr flows are treated and the excess (beyond the STP capacity) is overflowing at Dublin &

- ❑ For 10Yr, overflow the extra extra (untreated) at Beach Rd & Dublin St PSs only.

To “hit the target” only the following refinements were necessary.

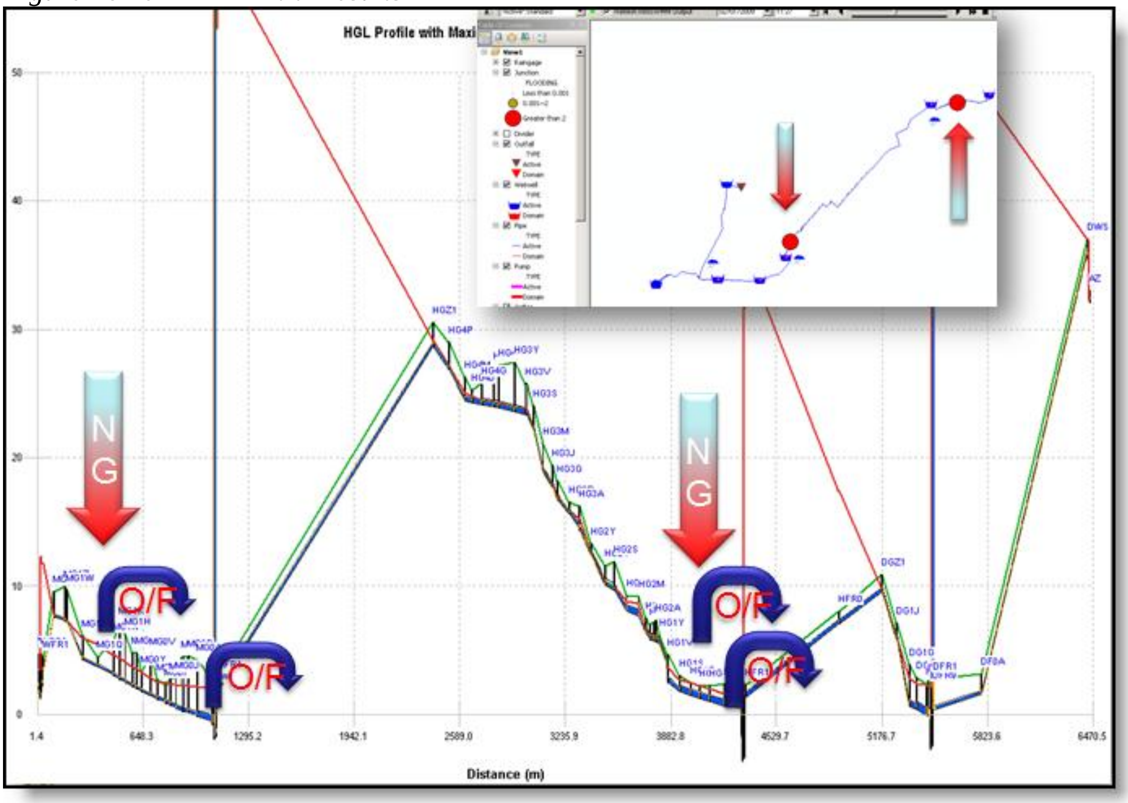
- Slightly Increase diameters of a few pipes
- Slightly Increase size of some pumps
- Enlarge bypass treatment
- Cap low Manholes U/S of Beach Rd PS
  (more details later)

3.4 20 YR ARI

The network was then checked for the 20Yr ARI using the same target as the 10 Yr ARI. (ie O/F only at BeachRd and DublinSt PSs). The I&I loading rates were determined by extrapolating the analysis curves, see Figure 18 RDII Rates & Total Load Respc .

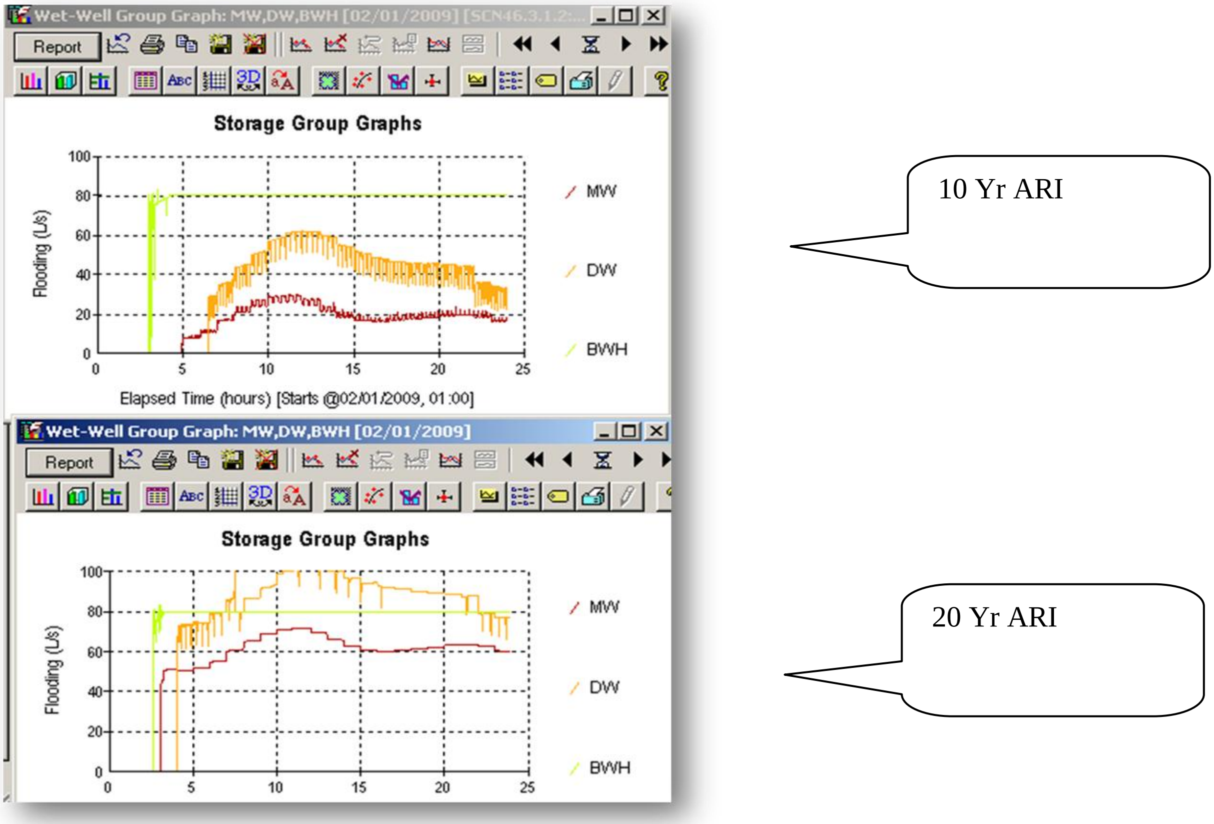
The initial results (no infrastructure changes) are shown below;

Figure 26 20Yr ARI initial results



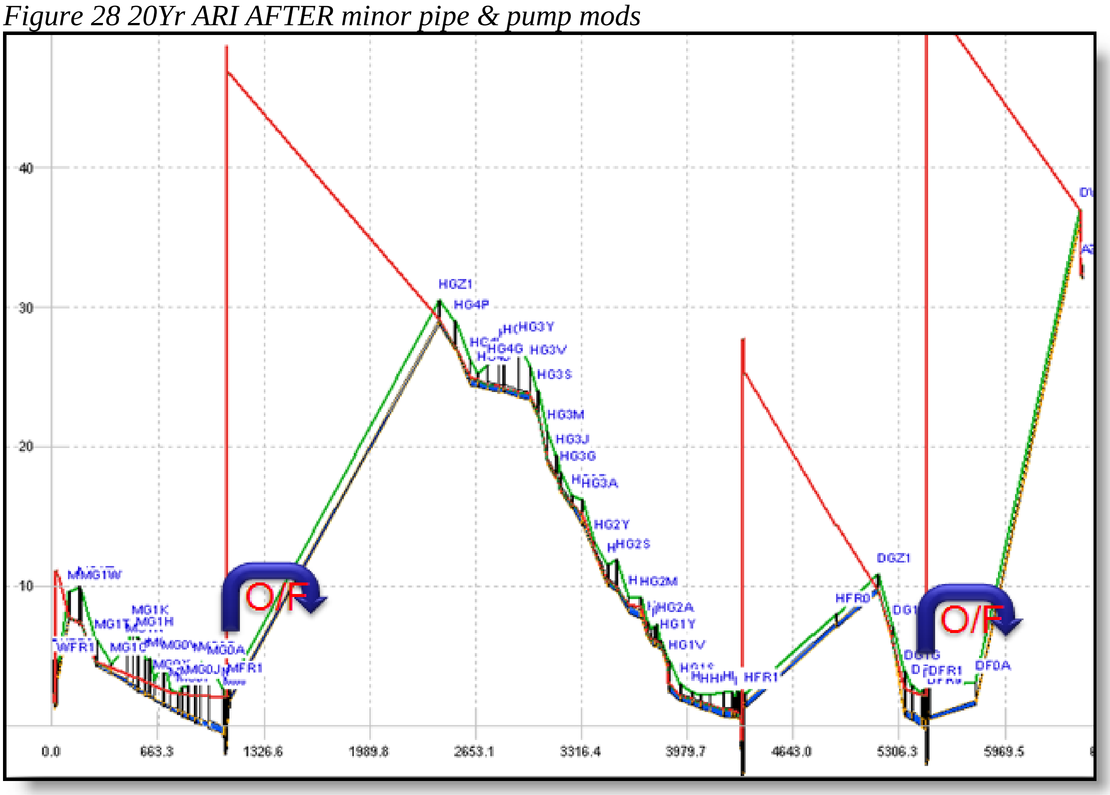
The figure above shows two new overflows [NG] along the pipeline (plus of course higher overflows at the pumpstations as illustrated in the figure below.)

Figure 27 Flooding (O/F) at Beach Rd (MW) & Dublin St (DW), & from the Bypass (BWH)



To eliminate the extra pipeline overflows, the Surrey St pump was increased slightly and one more low level manhole upstream of the Beach Rd PS was capped (thus forcing the overflow to occur only at the pumpstations - where environmentally it can be handled). The result is shown below;

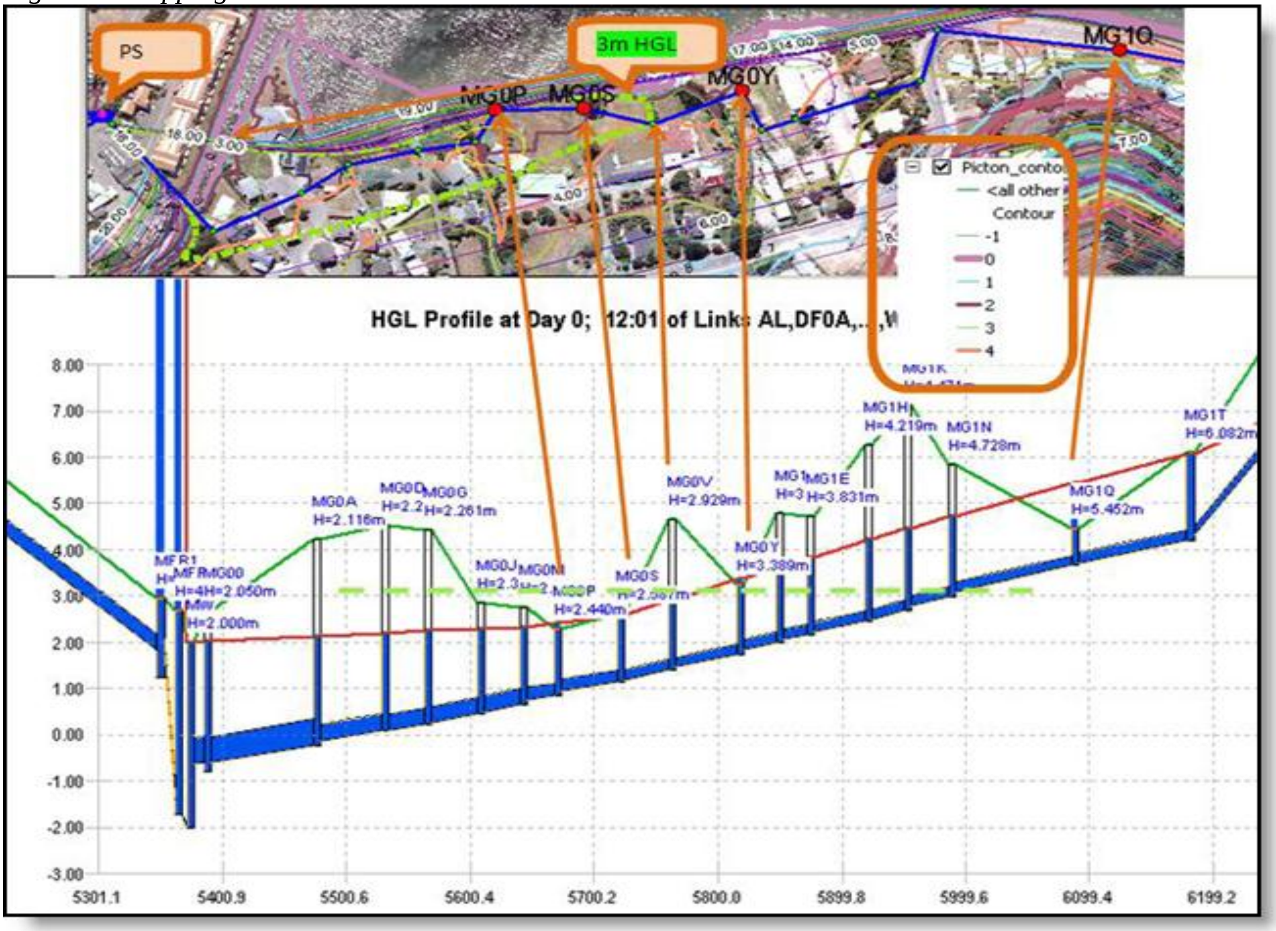
Figure 28 20Yr ARI AFTER minor pipe & pump mods



This is much better.

The capping details are shown below;

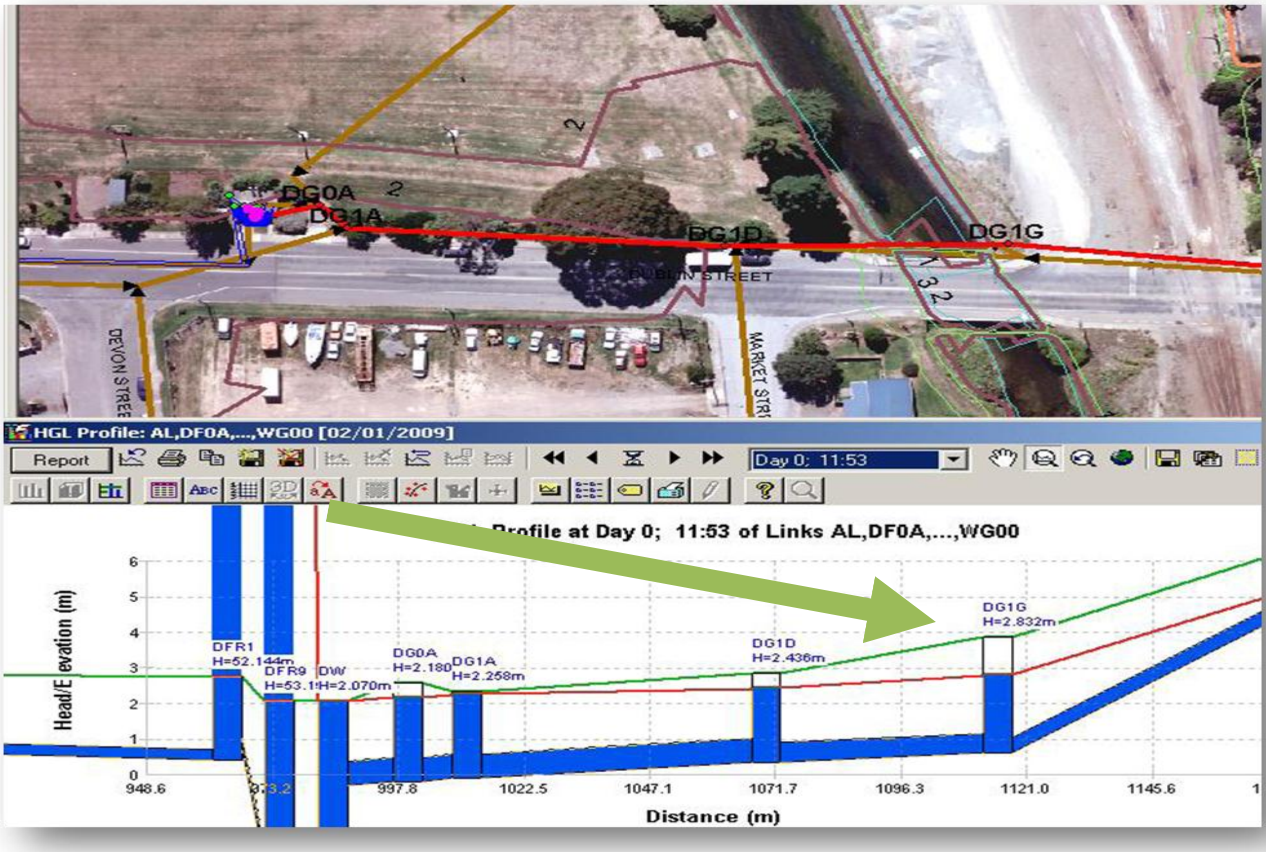
Figure 29 Capping details



All capped manholes are local low-spots (red dots in figure above) and all HGLs are comfortably below nearby floorlevels. This approach thus seem worthy of detailed evalution during detailed design as it extends the target so overflows from 20yr ARI loads can also be confined to designed overflow systems at the two *desired* pumpstations locations.

At the Dublin St PS site the detailed HGL is shown below.

Figure 30 Dublin St HGL



Because;

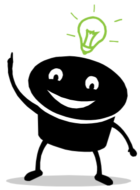
- the overflow will be directed into the nearby Waitohi Stm (for ARI ≥ 10yr) and

- the stream is likely to be **in flood** at the time,

then

- it may be necessary to **pump** the overflow.

An **alternative may be to make the low-lift bypass pumps larger** so the overflow occurs [Green arrow in figure above] after the pumps (and before the screens). Final decision will depend upon detailed PS design (&reticulation modelling).



4 CASE STUDY; SUMMARY

4.1 DATA ANALYSIS

The case study has shown that **RDII analysis to be a very helpful** way of assessing reality and estimating actual loads. It also provides a methodology of finding **realistic** metrics for daily domestic WW discharge [l/p/d] and associated peaking factors [PF] and *a way* of estimating dry weather GWI [however DW flows not very important beyond about a 6mo ARI (when they become over-shaded by RDII) .

The RTK **SUH methodology worked well**:

- o For determining mathematical **relationships** between rain events and [resulting] RDII and
- o For estimating **peak flows** for various [larger] events

and thus formulating a **simple stochastic loading set of RDII as l/s /ha.**

4.2 DESIGN

Using a stochastic loading set, enabled preliminary design to be refined to include the **management of overflows**
(ie "Plan to Overflow")

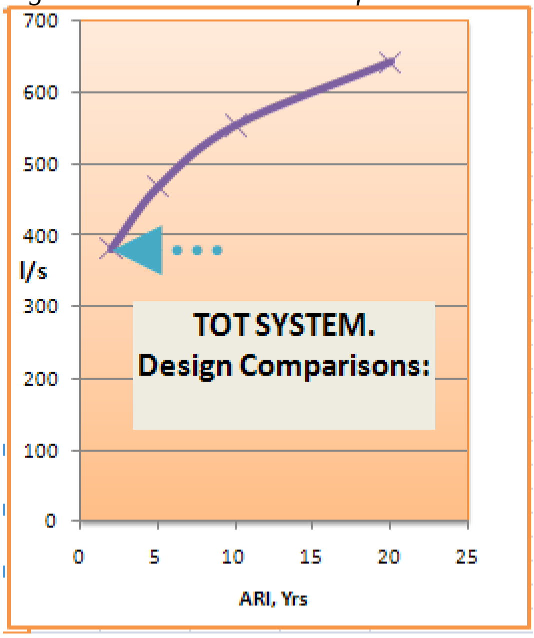
Modifications were slight but benefits large…. Thus, **very; cost and environmentally effective**

4.3 COMPARISONS

Comparison between classical load estimates and that which would be predicted by non-stochastic methods using actual data record revealed that:

- NZS4404 is not sufficient
- Using a fixed value for I&I loading also falls short (see figure below)

Figure 31 Stochastic Curve v fixed non-stochastic design load



5 CONCLUSION

I&I can be huge[j]; so the concept of a fixed design loading is not sufficient for catchments of the type studied nor does it convey the uncertainty & stochastic nature of overflows.

The following concluding comments are offered:

- The compromise in each area needs to match what can be practically, economically and environmentally achieved.
- This needs to be conveyed to, and trade-offs evaluated by, the community
- Basically it suggested that;

Overflows need to be: admitted, discussed with the community, and planned for.

And, the conclusion for the case study is that; stochastic analysis facilitated an initial design which *plans for overflows*. So; in answer to the question; ***how often will sewage overflow….?***

	Never[k] along the backbone trunk main
	No overflows below a 2yr ARI. All WW is contained, treated and discharged to the outfall diffusers
	Once[l] every 5 yrs treated[m] sewage will overflow, by design**,** into nearby **flooded** streams thence to open sea
	Once every 10yrs untreated sewage will overflow into nearby flooded streams then to, or direct to, open sea
	Even for a 20yr ARI overflows still only occur into nearby flooded streams
	Overflows, (especially for the 10yr & beyond case and when sea-level rises from climate change), **will need to be pumped** into the flooded streams/open-sea

[j] Eg 85% of the total peak load

[k] *up to the 20yrARI case evaluated

[l] On average (typical)

[m] Basically treated by Screens and UV

6 APPENDIX

6.1 APX1. RTK SUH BACKGROUND

Synthetics Unit Hydrograph (SUH) methodology using the RTK method was utilised for this analysis. This methodology was recommended in the study done for EPA in 2008. [Ref(5)]. The following are quotes from Chapter 4 thereof;

The RTK method, one kind of the SUH method, uses three triangular unit hydrographs to represent the various ways that precipitation contributes to RDII. The RDII volumes of three unit hydrographs are designated as R_1, R_2, and R_3. A high R_1 value indicates that the RDII is primarily inflow driven. If more of the total R-value is allocated to R_2 and R_3, this indicates that the RDII is primarily infiltration driven. This knowledge is useful during a sewer system evaluation survey..

The UH approach used in the RTK method is a common method for generating a hydrograph from a rainfall record based on linear response theory. One benefit of using a UH technique to determine rainfall responses in a sewer system is that the technique can be applied to analyze RDII flow from storms that have complex patterns of rainfall intensities and durations. The RTK method has been included as an option in SWMM4 and SWMM5 and has been widely used and proven to be a valuable method in separate sanitary sewer system analysis associated with storm events.

The following is from the EPA website; http://www.epa.gov/nrmrl/wswrd/wq/models/ssoap/

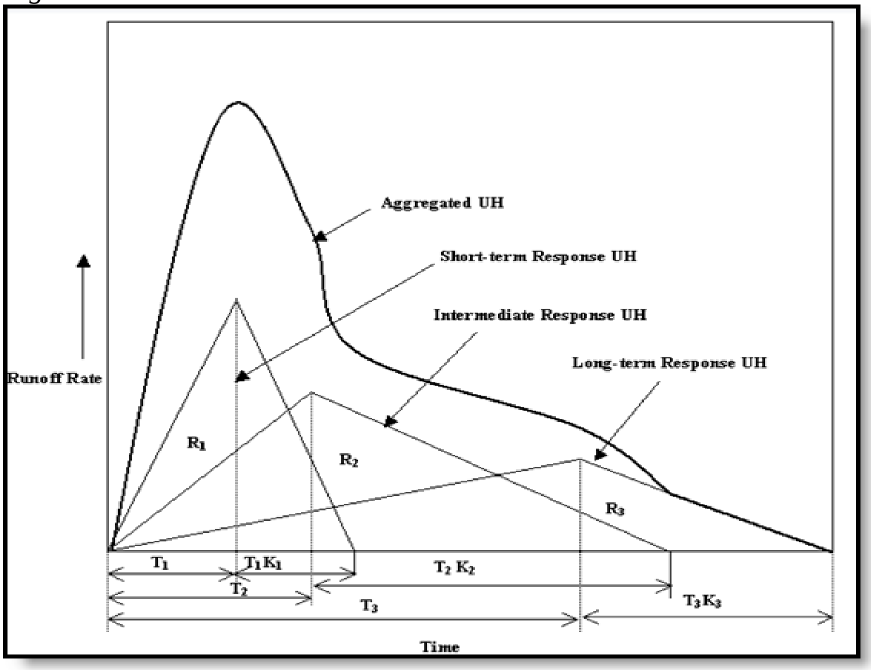
The RTK method is probably the most popular SUH method. This method is based on fitting up to three triangular unit hydrographs to an observed RDII hydrograph shown above to estimate the fast, medium, and slow RDII responses.

The R_i parameter is the fraction of rainfall volume entering the sewer system as RDII, T_i is the time to peak, and K_i is the ratio of time of recession to T_i.

The RDII volumes of three unit hydrographs are designated as R_1, R_2, and R_3. A high R_1 value indicates that the RDII is primarily inflow driven. If more of the total R value is allocated to R_2 and R_3, this will indicate that the RDII is primarily infiltration driven.

RTK is illustrated below. By definition the total R cannot be greater than 1 (*there can't be more inflow than rain*)

Figure 32 RTK SUH



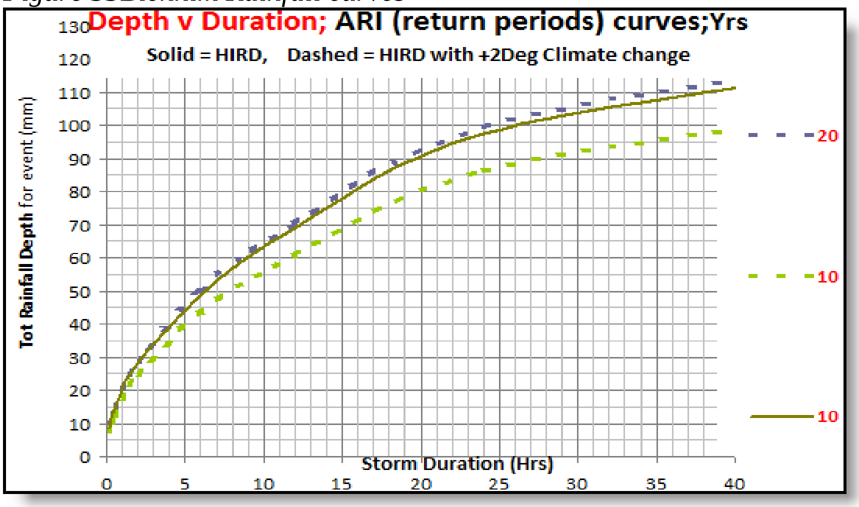
The following is from the Software documentation;

Traditionally, calibration of RDII UH parameters is performed through a tedious and inexact trial-and-error process in which the parameters are manually adjusted in an iterative fashion to closely match wet-weather flow data. Since there are a vast number of possible combinations of RTK values, evaluating all options this way may not be manageable, and even knowledgeable modelers often fail to obtain good results. RDII Analyst uses Genetic Algorithms optimization to automatically determine the UH parameters that best match the RDII time series generated by decomposing the measured flow data with the RDII flow estimated using InfoSWMM .

6.2 APX2. CLIMATE CHANGE

Using NIWA HIRD info (http://hirds.niwa.co.nz/) gives the following example of climate change effects. A 2deg C climate-change makes the 10 yr ARI curve essentially the same as the old 10 yr curve.

Figure 33Blenhim Rainfall curves



6.3 APX2. GLOSSARY AND TERMINOLOGY

Abbrev	Meaning
aka	also known as
ARI	Average Recurrence Interval (usually Yrs). aka Return-Period or RP
D/S	Down stream
DWF	Dry Weather Flow
EPA	Environmental Protection Agency (USA)
f(a,b)	Result is f(a,b) means that Result is a function of a & b
GWI	Ground Water Infiltration (in Dry weather)
HIRD	High Intensity Rainfall Design System
I/I, I&I, **or II**	Inflow and Infiltration (usually into the pipes of a WW system)
ID	Internal Diameter
infoSWMM	InnoVyze Software's implementation of SWMM
MDC	Marlborough District Council, New Zealand
MinF	Similar to PF but Minimum
NIWA	National Institute of Water and Atmospheric Research
PDWF	Peak Dry Weather Flow
PF	Peaking Factor (the ratio of peak hrly load from Domestic discharge to average dry weather flow)
PS	Pumpstation
PWWF	Peak Wet Weather Flow
RDBMS	Relational Data Base Management System
RDIIA	Rain Derived I&I Analysis -- also a module in SWMM and infoSWMM for same
RTK	A SUH Methodology. The names of the terms in same. Refer section *6.1*
SSOAP	Sanitary Sewer Overflow Analysis and Planning. Also a EPA software toolbox for SWMM (EPA) software.
STP	Sewerage Treatment Plant
SUH	Synthetic Unit Hydrograph
SWMM	Storm Water Management Model (EPA). Also the title of public domain software that Stormwater and WW analysis, by EPA
U/S	Up stream
UH	See SUH
VSD	Variable Speed Drive (eg for pumps)
WW	Waste Water also Wet Well
WWF	Wet Weather Flow

7 ACKNOWLEGEMENTS & REFERENCES

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1. The web site owners for pictures etc including those who's URLs are shown.
2. The author is indebted to the many clients, colleagues, practical engineers, operators and maintenance people who tried to teach me to *keep it real.* Some have succeeded <grin>.
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   Officially thought, the options and views expressed herein, are the authors and do not represent these organization's views or opinions.

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Syntax: Author, Title, Publisher

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