A Case Study on Urban Flood Modelling in Sydney using DRAINS and HEC-RAS

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Abstract

Flood mapping is a crucial element of flood risk management in both rural and urban development projects. In small and ungauged basins, empirical and regionalisation approaches are often adopted to estimate design flood hydrographs that represent input data into a hydraulic model. The limitations of observed runoff data in urban catchments in Australia present a major challenge with respect to direct model calibration and verification.This paper presents a case study on flood risk assessment in a small urban development project in Sydney, Australia. For this purpose, DRAINS and HEC-RAS models are adopted. It is noted that in Australia a more holistic approach of flood modelling (e.g. Monte Carlo simulation) is advocated in the recent edition of Australian Rainfall and Runoff (ARR), the national guide. However, there is limited data availability in applying such holistic approaches.

Keywords:HEC-RAS, DRAINS, floods, Monte Carlo simulation

INTRODUCTION

Flood destruction causes massive economic cost with major social disruptions associated with emotional disturbance, relocation, counselling, loss of important private and personal articles and in some cases loss of human life. For many urban development projects, flood risk assessment is needed to ensure unnecessary flooding of the property. The Statistical Rational method is the most common method, both in Australia and overseas for urban flood calculation. Although the hydrograph methods were not well known in 1960s, they still derived hydrographs using the Rational method. They produced triangular and trapezoidal hydrographs using the peak discharge from the Rational method and the time of concentration from other methods.

As reported in Aitken (1975), there had been many attempts to use overseas computer models directly or to modify overseas models to suit Australian urban catchments. Two problems were found in selecting overseas computer models for use in Australia. The first problem was the single systems used for urban stormwater and sewage water collection as opposed to a two separate system in Australia. The second problem was the soil types with high differences in infiltration in urban areas in different locationsof Australiacompared to the overseas models. Over the years, these minor problems were modelled to work around these two problems and suiting Australian conditions. An example of this is the ILSAX (O'Loughlin, 1993) model. Several well-knownurban drainage computer models are widely used in Australia and overseas with another popular model, HEC-RAS. HEC-RAS (Hydrologic Engineering Centre, 2000) is recognised as a successfulurban drainage computer model for design and analysis.

The Rational Formula method is simply a mathematical rainfall-peak runoff model using two ways, deterministic and statistical. Aitken (1975) found that the Rational Formula method as deterministic model was of almost no use in the urban situations. Statistical Rational Method/Model had some advantage needing personal judgement on selection of runoff coefficient. The ARR87 recommends the Statistical Rational Method with several major assumptions made, which are:

(i) The design storm is uniform in intensity over the catchment in both time and space,
(ii) The rainfall duration is equal to the time of concentration of the catchment,
(iii) The peak runoff is a fraction of the average rainfall rather than the residual after abstraction of losses,
(iv) The return period of the peak discharge is equal to that of the rainfall intensity, and
(iv) Rainfall runoff response is linear.

Statistical Rational Method is given by:

$$Q_{\text{peak}} = C_y IA/360 \tag{1}$$

where
Q_{peak} is the peak discharge (m^3/s),
C_y is the runoff coefficient corresponding to return period y,
A is the catchment area (ha), and
I is the average rainfall intensity (mm/h) of a storm with return period y and storm duration t_c (hours).

For single land-use catchments, losses are assumed to be the same for the whole catchment, making the runoff coefficient is a function of return period and fraction of imperviousness. If the catchment consists with different land-uses having different losses, then the area-weighted runoff coefficient should be computed (Argue, 1986). The time of concentration in urban catchments can be calculated by interpolating the collected rainfall intensity for the site from Australian Bureau of Meteorology against the information collected from the contoured catchment plan map/s, such as the distance water will travel, the roughness of the ground and the slope of the site. The main shortfalls of this Statistical Rational method are:

- The subjectivity of the catchment runoff coefficient (although there are guidelines given in ARR87 based on limited data),
- Uniformly distributed storms are rarely experienced over the catchment,
- Storms are not uniform in intensity,
- The return period of runoff and rainfall would rarely agree,
- The catchment time of concentration may be unknown or at best variable,
- It is applicable only to small catchments, and
- Only peak discharge can be estimated.

In ARR1987, the Probabilistic Rational Method (PRM) was recommended for use in south-eastern Australia. The runoff coefficient is an important component of the PRM, which can be estimated from the contour map in ARR87 volume 2. French (2002) noted that the isopleths of runoff coefficient in ARR1987 ignore watercourses. Pirozzi et al. (2009) and Rahman and Hollerbach (2003) evaluated the PRM and linked the runoff coefficient with catchment characteristics, but they achieved limited success. Rahman et al. (2011) noted that there is a lack of independent evaluation of the PRM and designers have poor knowledge about the uncertainty in design flood estimates obtained by PRM. The ARR RFFE model recommended in ARR2016 has no urban application module, i.e. it is applicable to natural catchments only(Rahman et al., 2016).

ILSAX and DRAINS are widely used by the local government authorities and consultants in Australia to design and analyse urban drainage systems. Thereareinadequate guidelines available to develop models for both gauged and ungauged urban catchments. The DRAINS manual (O'Loughlin, 1998) provides the information on how to assemble data to construct a model and some guidelines how to interpret results.

FLOOD ESTIMATION METHODS

Flooding is one of the most manageable of natural disasters, if flood prone areas are identified and suitable flood mitigation strategies are implemented. The most practical way of identifying flood prone areas and the effectiveness of flood mitigation strategies is by the application of mathematical models, which considers complex hydrological and hydraulic processes of these areas. The hydrologic models compute peak flows and/or flood hydrographs to minimise flood damage. Errors in peak flow estimation causeeither undersized or oversized infrastructure. For an efficient and economic urban drainage system design, it is important to estimate the design flows and/or flood hydrographs accurately. In dealing with urban drainage design, some cases, full flood hydrograph is not required. Simple peak flow design methods particularly Statistical Rational method are sufficient to design inlets, pipes, gutters and channels in locations where rainfall variability and/or storage effects can be neglected for small urban catchments.

In these design methods, it is assumed that the calculated peak discharge has the same average recurrence interval (ARI) as the design rainfall in the design methods (ARI neutrality concept). These peak flow design methods are simple mathematical models. In most cases, the design of urban drainage systems involves consideration of flood storage, permanent storage, off-channel storage, inter-drainage diversions and pumping installations and silting of drains. Knowledge is required for flood hydrographs instead of just flood peak. A full hydrograph can be obtained from the rainfall-runoff models such as ILSAX (O'Loughlin, 1993). It should be noted that ARI neutrality concept has been widely criticised (e.g. Rahman et al., 2002a; Loveridge and Rahman, 2018) and the ARR2016 has recommended Monte Carlo Simulation technique for design flood estimation (Weinmann et al., 2002); however, its application has not been widely adopted in practice. In this regard, regional Monte Carlo simulation technique can be useful such as the method proposed by Caballero and Rahman (2014).

It is necessary to estimate the model parameters and land-use parameters for rainfall runoff modelling in the urban catchments. The parameters include infiltration and depression storage and the characteristics of the catchment (impervious area, supplementary area and pervious area). The ideal method to determine model parameters is to calibrate the models using observed rainfall and runoff data but there is large cost associated with monitoring of these catchments and hence little observed data is available.

COMPARISONS OF HYDROLOGIC AND HYDRAULIC MODELS

Most urban catchment models use hydrologic and hydraulic computations based on loss modelling, overland flow routing and pipe routing in simulating the runoff response. Table 1 shows the different methods used in urban catchment models. In the loss modelling, storm loss for an event is defined as the amount of precipitation that does not appear as direct runoff. The storm loss includes moisture obstructed by vegetation, soil infiltration or retained by surface storage (depression). It can occur from both impervious and pervious surfaces. These losses can be modelled by four different loss components: (a) impervious area depression storage (impervious area initial loss); (b) pervious area depression storage (pervious area initial loss); (c) pervious area continuous loss, and (d) evaporation loss from both impervious and pervious surfaces. It can be assumed that in storm events hydrograph modelling, evaporation from pervious and impervious areas can be neglected, compared to other loss. The use of probability distributed loss modelling as proposed by Rahman et al. (2002b) has not been applied in urban applications; however ARR2016 recommends to consider stochastic nature of losses (e.g. Loveridge and Rahman, 2014).

Depression storage is a volume that must be filled prior to the occurrence of runoff on both pervious and impervious areas and can be considered as an initial loss. It represents a loss caused by interception and surface ponding. In storm event modelling, evaporation loss is insignificant and therefore the impervious area depression storage is assumed to be a constant in most urban drainage models. Typical values would be 0 to 2 mm for impervious area depression storage.

The pervious area depression storage is subject to infiltration and evaporation, though it is small losses. Therefore, the pervious area depression storage is also assumed to be constant in most urban drainage models. Typical values would be 2 to 10 mm for pervious area depression storage

Table 1: Modelling Methods Used in Different Models in Australia (Dayaratne, 2000)

Model	**Continuous or Event Model**	**Impervious and Pervious area Lumped or Seperated**	**Loss Model**	**Overland Flow Routing Method**	**Pipe Routing Method**	**Can Water Quality Parameters be Simulated?**	**Output**
ILSAX/ DRAINS	Event	Separate	Horton	Time-area	Manning's equation	No	Hydrographs at each pit can be modelled
HEC-RAS	Not applicable	-Manning's equation -Gradually varied flow	No	Can be generated water surface profile	Manning's equation	No	
CIVILCAD	Event	Separate	Horton	Time-area	Manning's equation	No	Hydrographs at each pit can be modelled
SWMM	-Event or -Continuous	Separate	-Horton -Green-Ampt	Nonlinear reservoir	Kinematic wave	Yes	Hydrograph at each pit

Numerous equations have been developed for modelling the process of water entry into soil from the surface at one point. Some are based on empirical equations to infiltration data; others use numerical solutions to complex equations.Other types of infiltration loss models are topographical lumped (different ground material)models, which are constant loss rate, initial loss-continuing loss, proportional loss, antecedent precipitation index and SCS curve procedure (Nandakumar et al., 1994). From these methods, initial loss-continuing loss, constant loss rate (i.e. runoff coefficient) and SCS methods have been used in urban drainage computer models. The model parameters of these types are estimated using the total catchment runoff. This method was found to be widely used due to its simplicity and ability to approximate catchment runoff behaviour (Nandakumar et al., 1994). The ILSAX model uses initial loss-continuing loss model with its continuing loss model by the Horton equation, which considers average conditions over the entire catchment.

CASE STUDY

The case study location is Auburn, New South Wales, Australia. A flood study was requested by the client's local Council to provide the required Finished Floor for the proposed development. IM Engineering & Accredited Certifier was instructed to carry out these services. The site is located at No.45 North Street, Auburn, NSW (Figure 1). The proposal is for a suspended single storey outbuilding/granny flat. The existing dwelling at the front of the property remains.

Figure 1: Topographical view of case study site, 45 North Street, Auburn, NSW

The purpose of the study was to establish the 100-year flood level for the proposed outbuilding development, set floor levels, check pre-development and post-development depth of flows and study the velocity-depth product effects.Stormwater requirements were obtained from Cumberland Council (Auburn City Council) and the NSW Government Department of Planning's 'Floodplain Development Manual'and identified as below:

1. A stormwater study is necessary to determine the 100-year ARI water surface level.
2. The impact of the development on 100-year ARI inundation levels on adjoining properties.
3. The required floodway width for conveyance of the 100-year ARI overland flow through site with a maximum allowable velocity-depth product of $0.4m^2/s$.
4. Minimum floors levels to be at least 500mm above the determined 100-year water surface level. Garage floor levels or driveway crests are to be 150mm above the determined level of inundation.

The hydrological analysis of the catchment was carried out using the DRAINS program. The 5, 10, 20, 30, 60 and 120 minute storm durations were used in the analysis. The catchment area is approximately 40.7 Ha (refer to Figure 2). This catchment area was assumed with a 50% blockage factor for a more realistic result. The 100-year peak overland flow along the front, inside and to the rear of the site is found to be 21.934 m^3/s as determined by the DRAINS (Figure 3). This value was then implemented in HEC-RAS.

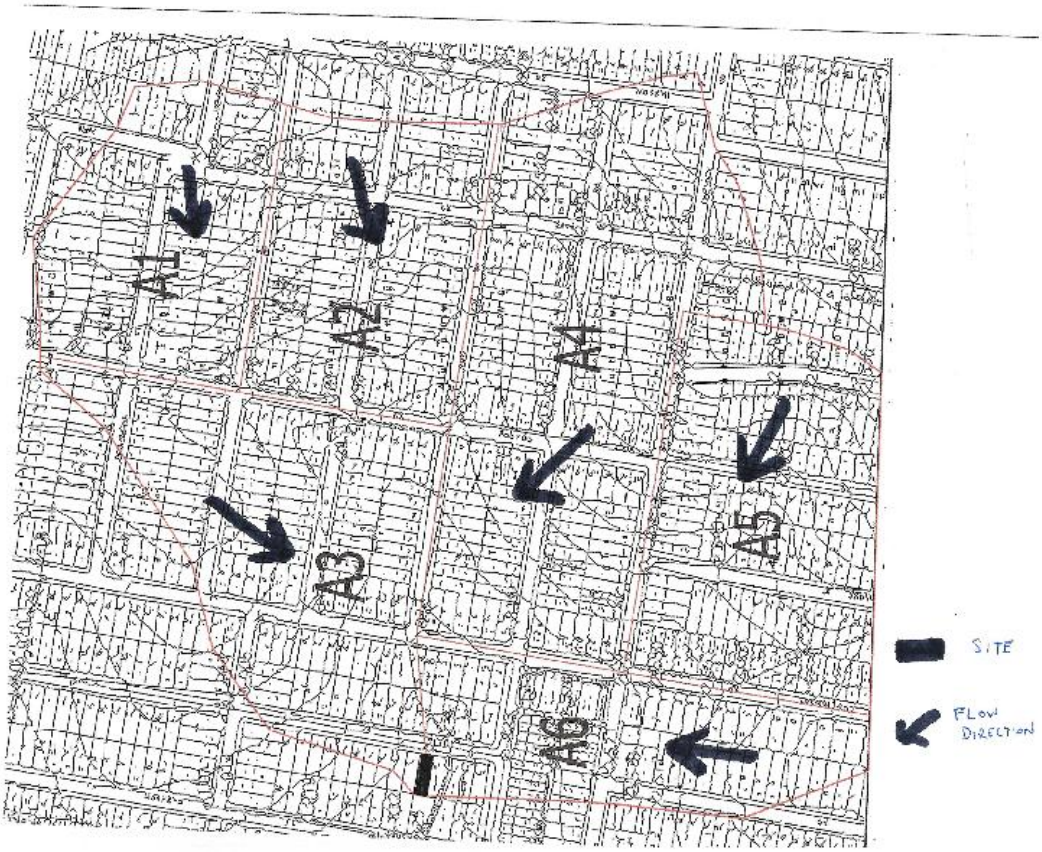
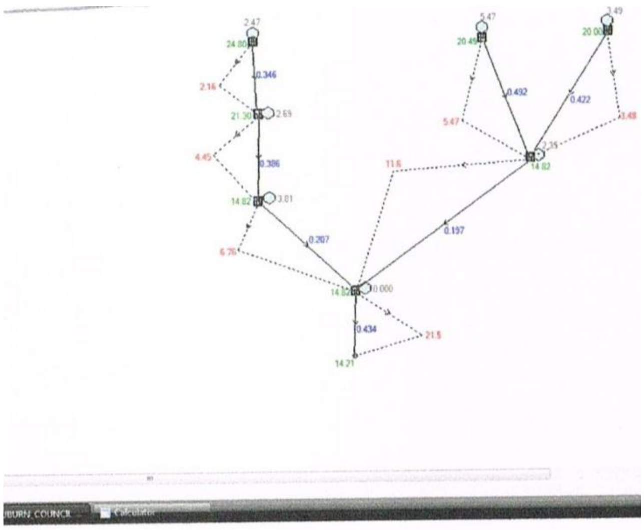
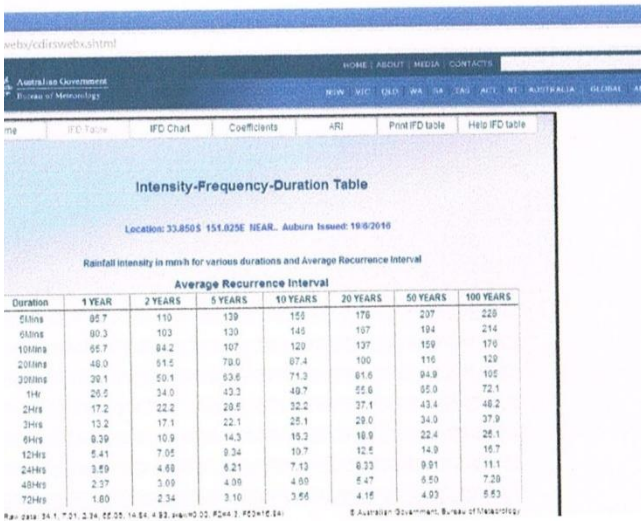


Figure 2: Catchment plan with sub-catchments to site for DRAINS modeling (Auburn Council, 2016)

The HEC-RAS model was used to analyse the pre-development and post-development status at 100-year flood levels. The pre-development analysis was based on existing levels across these properties. The post-development analysis was further based on the new finished levels proposed for the suspended granny flat.For each scenario (i.e. Pre-development and Post-development) an array of cross-sections were developed in modelling the area of interest. Additionally, obstructions and ineffective flows were also considered and included in each scenario modeled.





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Intensity-Frequency-Duration Table

Location: 33.850S 151.025E NEAR.. Auburn Issued: 19/6/2016

Rainfall intensity in mm/h for various durations and Average Recurrence Interval

Average Recurrence Interval

Duration	1 YEAR	2 YEARS	5 YEARS	10 YEARS	20 YEARS	50 YEARS	100 YEARS
5Mins	85.7	110	139	156	178	207	228
6Mins	80.3	103	130	146	167	194	214
10Mins	65.7	84.2	107	120	137	159	176
20Mins	48.0	61.5	78.0	87.4	100	116	129
30Mins	39.1	50.1	63.6	71.3	81.6	94.9	105
1Hr	26.5	34.0	43.3	48.7	55.6	65.0	72.1
2Hrs	17.2	22.2	28.5	32.2	37.1	43.4	48.2
3Hrs	13.2	17.1	22.1	25.1	29.0	34.0	37.9
6Hrs	8.39	10.9	14.3	16.3	18.9	22.4	25.1
12Hrs	5.41	7.05	9.34	10.7	12.5	14.9	16.7
24Hrs	3.59	4.68	6.21	7.13	8.33	9.91	11.1
48Hrs	2.37	3.09	4.09	4.69	5.47	6.50	7.28
72Hrs	1.80	2.34	3.10	3.56	4.15	4.93	5.53

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Figure 3: DRAINS model (version 2015.12) of sub-catchments for the site referring to rainfall intensity data for this site(http://www.bom.gov.au/hydro/has/cdirswebx/cdirswebx.shtml, 2016)

SCENARIO 1: Existing conditions i.e. pre-development

The computed maximum water depth from North Street shows that overland flows and/or any back flow was apparent. The highest maximum Water Surface Level (W.S.L.) achieved for the site is 16.11m A.H.D (Cross section 3) (Figure 4). With the existing development having a FFL of 15.49m, this indicates that the existing development will be flooded in the 100-year ARI flood event of a flood height of 0.62m to the top of water level.

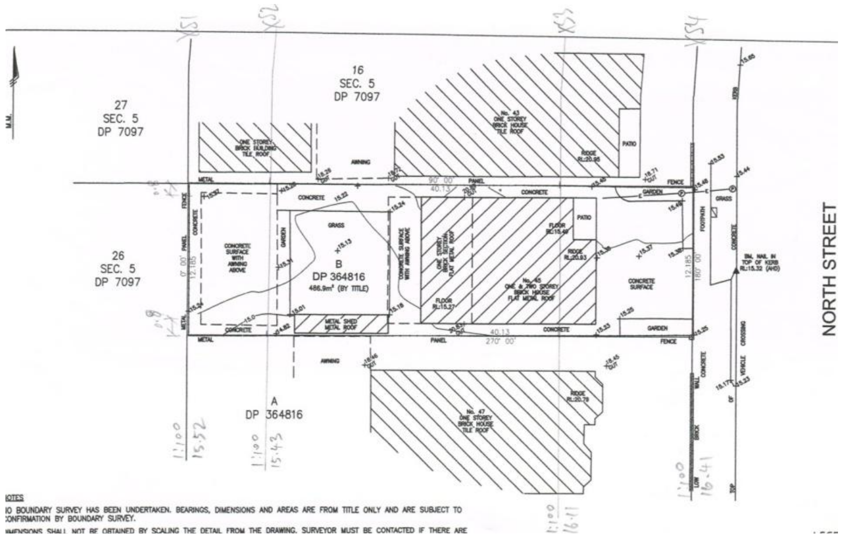

Figure 4: Existing site conditions (RGM Property Surveys,2016)

SCENARIO 2: Proposed conditions i.e. post-development

The post-developed scenario shows that if the proposed Granny Flat (Cross section 2) was filled under the ground floor to support the proposed ground floor slab, the water level will rise from 15.43m AHD (Existing flood level) to 15.72m AHD (Figure 5). This flood height will be 0.87m to the top of water level. By suspending the proposed ground slab to sit on a water-proofed and galvanised square hollow section posts sitting on concrete piers, this will make the flood levels at 'No Change'from the pre-developed stage. Cross-section 2 have a 100-year ARI flood level of 15.43m as the existing pre-developed stage shows the maximum height at the lowest point is 0.58m to the top of water level.

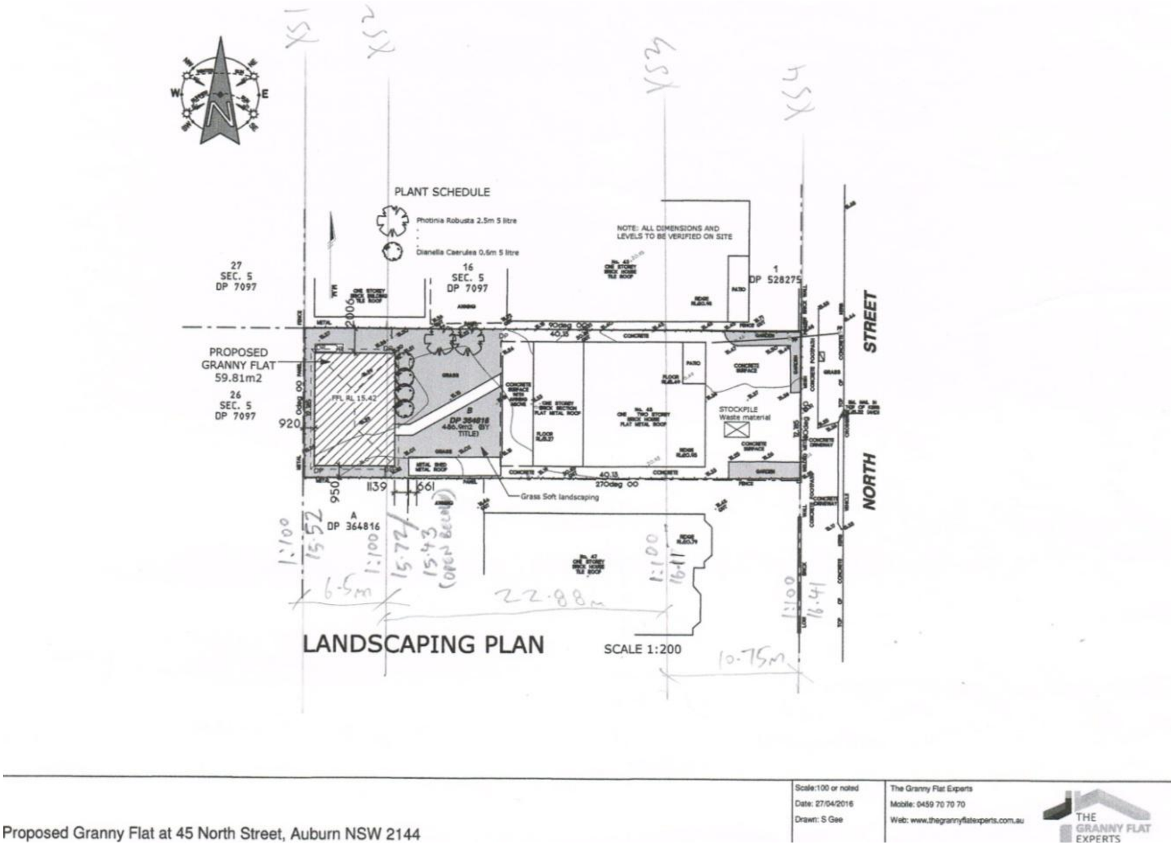

Figure 5: Proposed site conditions (The Granny Flat Experts, 2016)

From the above results, it can be concluded that the proposed solution will have noeffect on the existing neighbouring residences and on the future planning for this proposed development.

The no change of impact on the proposed granny flat on the 100-year flood levels were assessed (Refer to HEC-RAS summary table). It can be seen from the HEC-RAS results and sections modelled, that the proposed addition had no impact on the 100-year post-development flood levels even to the adjoining properties. The resulting 100-year flood levels are a conservative estimation and will actually be lower if the modelled flow width was extended (Figure 6).The 100-year water surface levels are as follows (refer to Table 2).

It is recommended that the floor level of the future building shall be a minimum of 500mm above the top of water level in order to maintain Council's 300mm freeboard requirement, plus an additional 200mm for any inaccurate assumptions made in the model.

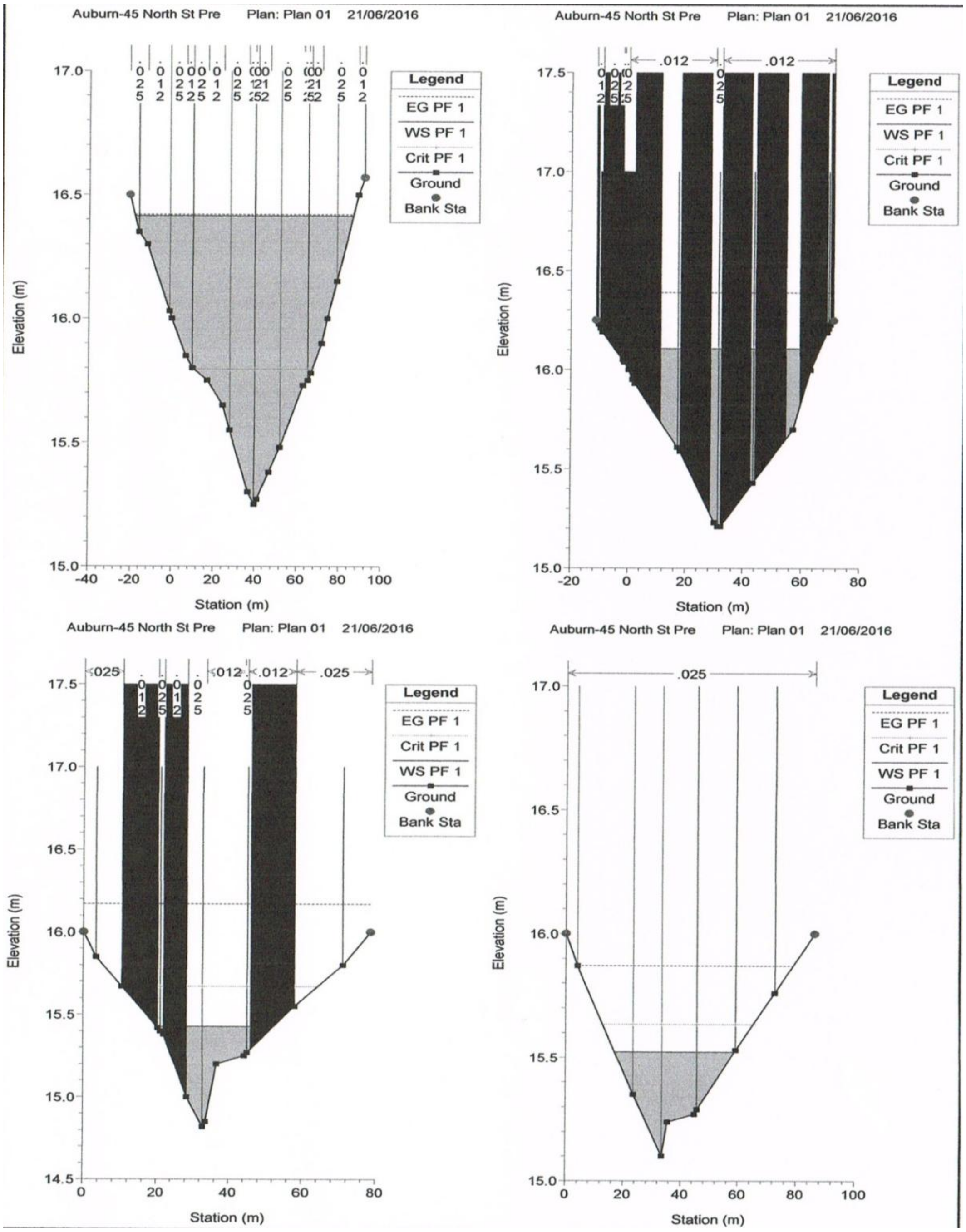

Figure 5: HEC-RAS 4.1.0 cross-sectional outcomes for PRE-DEVELOPED stage (www.hec.usace.army.mil/software/hec-ras, 2016)

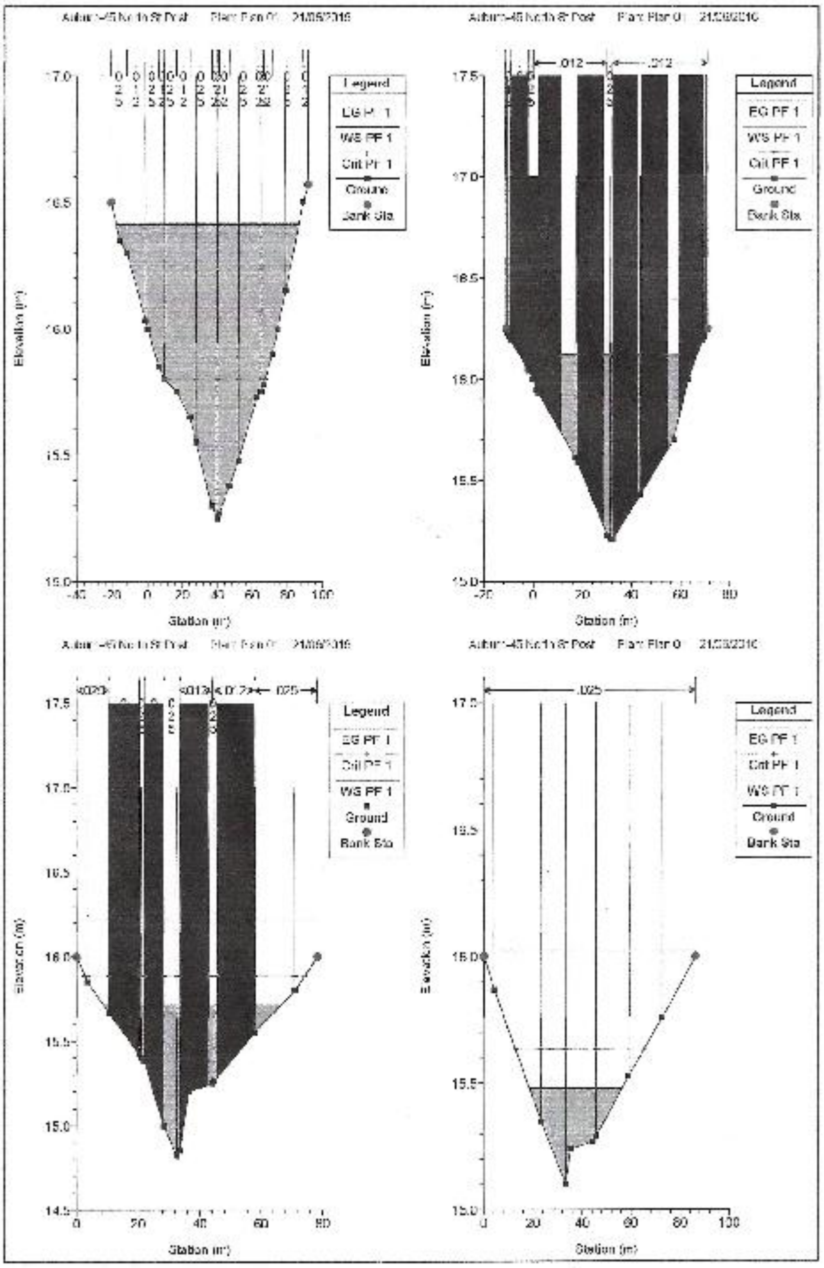

Figure 6: HEC-RAS 4.1.0 cross-sectional outcomes for POST-DEVELOPED stage (www.hec.usace.army.mil/software/hec-ras, 2016)

Table 2: HEC-RAS results with minimum required finished floor levels

PRE-DEVELOPMENT

River Station	Flood Level (m)	Minimum Required Floor Level (m)
4	16.41	-
3	16.11	16.61
2	15.43	-
1	15.52	-

POST-DEVELOPMENT:

River Station	Flood Level (m)	Minimum Required Floor Level (m)
4	16.41	-
3	16.11	16.61
2	15.43	15.93
1	15.52	-

CONCLUSION

The main aspects of an urban drainage system consist of property drainage, street drainage, trunk drainage and major water receiving bodies. In most drainage systems, retention and detention basins are also used for flood control and water quality improvement. Several urban drainage models have been developed to simulate the rainfall-runoff process of urban drainage systems. The major components of these models include the modelling of rainfall excess, overland flow routing and pipe routing. Different models use different methods to model these components. Most drainage models calculate rainfall excess using hydrologic methods and this rainfall excess is then routed through the pipe system and other system components using hydraulic methods. However, there are other models where hydrology and hydraulics of the system are lumped together in computing flood hydrographs and/or peak discharges. The choice between the two types of models depends on the type of the catchment to be modelled, the availability of catchment data, the level of complexity and sophistication required in the simulation of the catchment runoff response and time available for the analysis. There has been little advancement with respect to modelling method, although application of GIS has enhanced model visualization aspects.ARR2016 advocates application of Monte Carlo simulation approach for flood hydrograph modelling, which, however, is not practiced as yet in urban flood modelling. Hence, the uncertainty in flood level prediction is not fully assessed with the current deterministic approach.

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