Flood Risk report of River Irvine

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Enginyeria de la construcció

Barcelona, juny del 2018

Departament d'Hidràulica, Marítima i Ambiental

TREBALL FINAL DE GRAU

Agraïments

Després d'un període intens escrivint aquest treball, per fi és el dia: escric aquest apartat d'agraïments per tal de finalitzar el meu treball de fi de grau.

Als meus estimats pares i germà, per haver-me guiat i recolzat en totes les decisions de la meva vida i haver fet de mi la persona que sóc ara.

Al meu millor amic Óscar, per escoltar-me i aguantar-me sempre, i per ser la única persona capaç de tranquil·litzar-me en els moments més crítics.

Al meu yayo Ramón, qui hagués estat orgullós de tenir finalment una néta enginyera.

A la Raquel Salvador, per haver sigut el mirall en el que mirar-me i per transmetre'm els seus valors.

Als meus companys d'universitat Óscar, Martín, Tomás, Marc, Nico, Roger, Diegos, Karen, Pablo i Iván, perquè vam començar aquesta aventura junts i per fi ho hem aconseguit.

A en Daniel Ninyerola, per haver-me aconsellat i estat sempre disponible i fer que aquest treball hagi sigut possible.

A en Martí Sánchez Juny, per donar-me l'empenta final d'aquest treball de fi de grau.

Finalment a el Dr. Manousos Valyrakis, per motivar-me a fer aquest treball i haver-me ensenyat tantes coses.

A tots vosaltres, gràcies.

Laia Mestres Pi

Barcelona, 15 de Juny del 2018

SUMMARY

The impacts of flooding experienced by people, communities, property, land and businesses can be calamitous and long lasting. In Kilmarnock area (Scotland) for example, the most recent flooding occurred in December 2014. This affected properties along the River Irvine and Kilmarnock. The river Irvine, Kilmarnock Water and Back Burn are also known to have flooded in 2008, which mainly affected the Riccarton and Newmilns area in Kilmarnock. On July 2007 the River Irvine burst its banks in Newmilns, and flash floods affected roads including the M77. The Back Burn also overtopped on High Street where 25 residential properties and businesses were affected by the flood waters. The urban area of Kilmarnock is estimated to spend an average of £1.2 million per year in damages caused by flooding events.

The knowledge of the potentially flooding areas become a key factor in order to reduce any such future event, by means of land-use planning or activating managing risk mechanisms like emergency plans or prevention actions, and improve Scotland's ability to manage and recover from any events which do occur.

The present study has as general objective to build, run and analyze results of hydraulic models for different hydrologic scenarios in order compute and locate flood risk and identify areas with increased risk in Kilmarnock.

The specific objectives of this study are: to determine the design flow for different returning periods in order to define the potentially flooding areas using the software HEC-HMS, to determine the flooding areas and provide a final flooding maps using HEC-RAS and AutoCAD Civil 3D.

RESUMEN

El impacto de las inundaciones experimentados por personas, comunidades, terreno y comercios pueden ser devastadores y duraderos. En la región de Kilmarnock (Escocia) por ejemplo, el episodio extremo de inundaciones más reciente tuvo lugar en Diciembre del 2014. Éste afectó numerosas propiedades a lo largo del río Irvine y Kilmarnock. El río Irvine, Kilmarnock Water y Back Burn también se conoce han sufrido inundaciones en 2008 que afectaron principalmente las zonas de Riccarton y Newmilns en Kilmarnock. En Julio 2007 el río Irvine se desbordó en Newmilns e inundaciones repentinas afectaron numerosas carreteras incluyendo la M77. El Back Burn también superó sus límites en High Street donde 25 propiedades residenciales y comercios fueron afectados por las inundaciones. Se estima que en el área urbana de Kilmarnock se invierten un promedio de £1.2 millones al año en daños provocados por episodios de inundaciones.

El conocimiento de las áreas potencialmente inundables se convierte en un factor clave a fin de reducir cualquier evento futuro, por medio de ordenación territorial o activando mecanismos de gestión de riesgos como planes de emergencia o acciones preventivas, y mejorar la capacidad de Escocia de gestionar y recuperarse de cualquier evento que ocurra.

El presente estudio tiene como objetivo definir, ejecutar y analizar los resultados del modelo hidráulico para diferentes escenarios hidrológicos a fin de calcular y localizar el riesgo de inundación de las áreas con mayor riesgo en Kilmarnock.

Los objetivos específicos del estudio son: determinar el caudal de diseño de la avenida para diferentes periodos de retorno para definir las zonas potencialmente inundables usando el programa HEC-HMS, determinar las áreas de inundación y proporcionar los mapas de inundación usando HEC-RAS y AutoCAD Civil 3D.

RESUM

L'impacte de les inundacions experimentades per persones, comunitats, terreny i comerços poden ser devastadors i de llarg termini. A la regió de Kilmarnock (Escòcia) per exemple, l'episodi extrem d'inundacions més recent va tenir lloc al Desembre del 2014. Aquest va afectar nombroses propietats al llarg del riu Irvine i Kilmarnock. El riu Irvine, Kilmarnock Water i Back Burn també es coneix que han patit inundacions al 2008 que principalment van afectar les zones de Riccarton i Newmilns a Kilmarnock. Al Juny del 2007 el riu Irvine es va desbordar a Newmilns i les inundacions sobtades van afectar a nombroses carreteres incloent la M77. El Back Burn també va sobrepassar els seus límits a High Street on 25 propietats residencials i comerços van ser afectats per les inundacions. S'estima que a l'àrea urbana de Kilmarnock s'inverteixen un promig de £1.2 milions a l'any per danys provocats durant episodis d'inundacions.

El coneixement de les àrees potencialment inundables esdevé un factor clau a l'hora de reduir qualsevol esdeveniment futur, mitjançant la ordenació territorial o activant mecanismes de gestió de riscos com plans d'emergència o accions preventives, i millorar així la capacitat d'Escòcia de gestionar i recuperar-se de qualsevol esdeveniment que es produeixi.

El present estudi té com a objectiu definir, executar i analitzar els resultats del model hidràulic per diferents escenaris hidrològics per tal de calcular i localitzar el risc d'inundació de les ares de major risc a Kilmarnock.

Els objectius específics de l'estudi són: determinar el caudal de disseny de l'avinguda per diferents períodes de retorn per tal de definir les zones potencialment inundables utilitzant el programa HEC-HMS, determinar les àrees d'inundació i proporcionar els mapes d'inundació utilitzant HEC-RAS y AutoCAD Civil 3D.

PROJECT REPORT

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East Ayrshire Council, located in London Road in Kilmarnock (East Ayrshire) requests the flood risk study for the Kilmarnock area and to prepare a Flood Risk Assessment in line with the latest policy documents (Flood Risk Assessment Planning Guidance for Developers: Glasgow City Council, May 2011 and Scottish Planning Policy 2014). The exact location of the area of study can be consult on the drawings.

According to the Scottish Planning Policy (SPP, 2010 and draft SPP of April 2013), flood risk assessment is provided through the Risk Framework, differentiating between probability, hazard, and vulnerability. Annual flood probability forms the basis for characterising areas for planning purposes. Probability of flooding is assessed upon areas of little or no risk (0.5% or 1 in 200 years returning period) and corresponding planning restrictions and guidance provided. The design criterion for assessing hazard in Scotland is specified as a minimum of 1 in 200 years returning period.

The indications provided in the following report are coherent according to what is determined by the SPP7: Planning and Flooding approved the 4th of February 2010, specially to the section *The planning approach to assessing risk*. In allowance with this section, point 33 is worded as follows:

> *"33. The probability of any site being flooded lies between virtually zero (0.0%) and near certainty (100%). Even in areas generally free from flooding, local conditions and exceptional rainfall can lead to flooding. It is therefore not possible to set planning policy and determine applications solely according to the calculated probability of river or coastal flooding. Nevertheless, to provide a basis for decision making, a characterisation of flood risk into 'little or none', 'low to medium' and 'medium to high' is set out in the Risk Framework even though this necessarily simplifies the situation. For each level of risk an appropriate planning response is outlined."*

In the Risk Framework the characteristics of each area are detailed in addition to its appropriate planning response.

> *"1. <u>Little or no risk area</u> - Annual probability of watercourse, tidal or coastal flooding: less than 0.1% (1:1000). <u>Appropriate Planning Response</u> - No constraints due to watercourse, tidal or coastal flooding. "*

"2. <u>Low to medium risk area</u> - Annual probability of watercourse, tidal or coastal flooding: in the range 0.1% to 0.5% (1:1000 to 1:200). <u>Appropriate Planning Response</u> - [...] Suitable for most development. [...] Subject to operational requirements, including response times, these areas are generally not suitable for essential civil infrastructure such as hospitals, fire stations, emergency depots etc. Where such infrastructure has to be located in these areas or is being substantially extended, they must be capable of remaining operational and accessible during extreme flooding events."

"3. <u>Medium to high risk area</u> - Annual probability of watercourse, tidal or coastal flooding: greater than 0.5% (1:200). Generally not suitable for essential civil infrastructure, such as hospitals, fire stations, emergency depots etc. schools, ground based electrical and telecommunications equipment. The policy for development on functional flood plains applies. Land raising may be acceptable."

Furthermore there is a subdivision in the 'medium to high risk area' depending if the area is developed (build-up) or not at the moment of the study. This characterization is presented below:

"3.(a). <u>Within areas already built-up</u> - <u>Appropriate Planning Response</u> - These areas may be suitable for residential, institutional, commercial and industrial development provided flood prevention measures to the appropriate standard already exist, are under construction or are planned as part of a long term development strategy in a structure plan context. In allocating sites preferences should be given to those areas already defended to that standard. Water resistant materials and construction as appropriate."

"3. (b). <u>Undeveloped and sparsely developed areas</u> - <u>Appropriate Planning Response</u> - These areas are generally not suitable for additional development, including residential, institutional, commercial and industrial development. Exceptions may arise if a location is essential for operational reasons, e.g. for navigation and water-based recreation uses, agriculture, transport or some utilities infrastructure, and an alternative lower risk location is not available. Such infrastructures should be designed and constructed to remain operational during floods. These areas may also be suitable for some recreation, sport, amenity and nature conservation uses (provided adequate evacuation procedures are in place). Job-related accommodation (e.g. caretakers and operational staff) may be acceptable. New caravan and camping sites should generally not be located in these areas. Exceptionally, if built development is permitted, flood prevention and alleviation

measures are likely to be require and the loss of storage capacity minimized. Water resistant materials and construction as appropriate. Land should not be developed if it will be needed or have significant potential for coastal managed realignment or washland creation as part of an overall flood defense.

Additionally point 36 establishes the addition of an allowance for climate change when required:

"36. In these circumstances it is not national policy to add an additional allowance for climate change above 0.5% probability but planning authorities may do so if it can be justified, taking account of the most recent UKCIP scenarios as applied to the area concerned. A freeboard allowance may be required as a response to local circumstances."

Despite the applicability of the previous Scottish regulations, there is no either Scottish or UK regulations for defining a buffer zone method, which would be very useful in this case in order to elaborate the risk assessment. For this reason, Spanish regulations of Public Domain will be overlap together with the Scottish regulation obtaining a complete risk assessment.

Considering the latest modification of the Hydraulic Public Domain Regulation collected in the RD 9/2008 , the flood plains corresponding to 10, 50, 100 and 500 years returning period are determined. The indications complied below are according to what is determined in the decree 305/2006 of the Generalitat de Catalunya from which the Urban Planning Regulation is approved, particularly its 6th article, Guidelines for preservation against flood risk. Seeing the last modification of the 6th article from Hydraulic Public Domain Regulations, in its section 2 is worded as follows:

"2. The protection of the hydraulic public domain has as its main objectives the ones listed in the 92nd article from the Waters Law text. Unprejudiced with the specific techniques committed to carry out the accomplishment of the already said goals, the edges of the field which border the riverbed are subjected in all its longitudinal length:"

*a) An **area of servitude** of 5m width for its public use, which is regulated in this regulation.*

*b) A **police area** of 100m width, where the use of the ground and the activities developed there will be regulated.*

In section 7 the characteristics of the **area of servitude** are detailed, among them the following ones stand out:

"3. In general no construction will be done in this area excepting if it is convenient or necessary for the use of the hydraulic public domain o for its conservation or restoration..."

In section 9 the characteristics of the **police area** are detailed, among them the following ones stand out:

"1. In the police area with 100m of width measured horizontally from the riverbed are subject to what s provided in this Regulation the following activities and uses of the land:"

...

d) Any other use or activity that could involve an obstacle for the flow in floods regime or that could cause degradation or deterioration of the mass of water state, of the aquatic ecosystem, and in general, of the hydraulic public domain.

Finally, in section 14 of the Hydraulic Public Domain Regulation the characteristic of the **flooding areas** are defined:

"1. Are considered as flooding areas those ones limited by the theoretical levels that the flood water will reach for 500 years returning period, according to geomorphologic studies, hydrological and hydraulic, as well as series of historical floods and documents or evidences of the same, unless the Ministry of Environment, on proposal of the basin organism fixes, on concrete file, the delimitation that would fit better in each case according to flow behavior. The qualification as flooding areas will not alter the legal qualification and the ownership that each land had."

In order to check that each one of the points above are accomplished according to the Scottish regulations, the hydrological characteristics of the basin, downstream the study area, and then the hydraulics calculations have been done.

2. Hydrological Study

In the previous section it has been explained the need of having the flow rate for the different returning periods of 10, 100 and 500 years. In order to obtain these flow rates it has been found in first instance the maximum flow rate.

The maximum flow rate has been calculated using the software Hec-HMS. This software is designed for modelling the most common hydrological processes happening on a river basin.

2.1. Applicability conditions

The methods that will be used from now on have a clear applicability conditions:

- The river basin cannot be total or mostly urban.
- The surface of the river basin (S) cannot be higher than 1000km^2 and the concentration time C_t cannot overcome 24h.
- The length of the section cannot be longer than 4km.
- Open channel flow conditions in all the length, with exception of the drainage works infrastructures that can cross the channel. Therefore, that sections of the river with coverings or burying are excluded.
- That the hypothesis of one-dimensional flow is valid.
- That the hypothesis of a fixed river bed is valid. That is to say that the changes that suffers the river bed due to flooding cannot be high enough to invalidate the results obtained using the fixed bed river consideration.

2.2. Definition of the river basin

Since the area of study is corresponding to the river Irvine's basin, four sub-basins have been defined:

- River Irvine's sub-basin: This sub-basin belongs to the North area of the river Irvine. This sub-area has been defined from the west side of the junction between river Irvine and Kilmarnock until 11km from the junction. It has been defined a width of 1.5km which is three times the maximum width of the river. The total surface of this sub-basin is 17.83km^2 and located in a latitude and longitude of 55.60429 and -4.410855 degrees.
- Kilmarnock's sub-basin: Belonging to the zone where the river is crossing Kilmarnock's urban area. This sub-basin has been defined from the North part of the junction between river Irvine and the river from Kilmarnock until Dan Castle Country Park's area. The width is 4.00km which is three times the maximum width of the river. The total surface is 14.66km^2 and it is located in a latitude and longitude of 55.617521 and -4.491508 degrees.
- Ayrshire's sub-basin: This sub-basin belongs to the Ayrshire's rural area. This last has been located in the junction between the tributaries and river Irvine. The width is 0.6km which is area of servitude of the tributaries. The total surface is 0.66km^2 and located in a latitude and longitude of 55.616615 and -4.447105 degrees.
- Final junction's sub-basin: Belonging to the union between the water from Kilmarnock and river Irvine until the end of the Kilmarnock's urban area. The width is 2.5km, the area is 2.62km^2 and it is located in a latitude and longitude of 55.596175 and -4.511829 degrees.

2.3. Basin scheme definition

Once defined the elements that will take part on the basin modeling, first of all it has been placed on the board river Irvine and Ayrshire sub-basin. Through a union, both sub-basins have been put together downstream. This junction will be used in order to perform the sum of the output flow hydrographs of each sub-basin. By propagation river Irvine has been united with its tributaries until the junction of river Irvine with the river from the urban area of Kilmarnock. Same way than before, the sum of hydrographs has been done.

Figure 1. Basin distribution HEC-HMS

For the basin calculation the corresponding Losses Method has been used, which is the modelling of each SCS Curve Number (SCS-CN) and will be explained in section 2.6. Then the Transformation Method will develop the corresponding hydrograph for each of the SCS Curve Number.

2.4. Definition of each subbasin

Each sub-basin is characterized in terms of length, width, area and location coordinates. Furthermore hydrographic units such as retention time (Lag time), SCS curve number, impermeability and initial infiltration are defined.

Name of the subbasin	Length (Km)	Average Width (Km)	Area (Km^2)	Latitude (Degrees)	Longitude (Degrees)	Lag time (min)	Curve Number (mm/h)	Impermeability (%)
Subbasin Kilmarnock Water	3,6646722	4,00	8,941800168	55,617521	-4,491508	25,51270816	42,625	18,37
Subbasin Irvine River	11,884716	1,5	28,99870704	55,60429	-4,410855	75,47022007	59,27	5
Subbasin Ayrshire Water	1,08	0,612	0,66	55,615716	-4,45671	10,49767485	67,625	5

Table 1. Subbasin geographic definition

In order to calculate the value of Lag time the following formula is used:

$$Lag\ time = 0.35 * C_t$$

To find the concentration time (C_t) it is used Muskingum's Method which calculates the time required for the kinematic wave to travel from end to end of the section of study. This parameter is known as K of Muskingum and it is calculated with the formula below:

$$K = 0.18 * (\frac{x}{i^{0.25}})^{0.76}$$

Where:

- "x" is the maximum distance that the water will flow the water inside the sub-catchment [km]
- "i" is the average slope of the sub-basin calculated as:

$$i = \frac{river\ slope\ (Km)}{maximum\ river\ distance\ (Km)} = \frac{h_i - h_f}{x_i - x_f}$$

Once K is calculated, the following formula has been used for the calculation of C_t and Lag time afterwards:

$$C_t = \frac{K}{0.6}$$

2.5. Rainfall data

For obtaining the flow rate of the probable maximum flood (PMF) it has been defined a new Gage through a Time Series Data Manager. For this it is necessary to open a new window of chronologic time defining the rain interval. IDF curves are required for this step.

An IDF curve or Intensity-Duration-Frequency curve is a mathematical relation, generally empiric, between the rain intensity, it duration and the frequency observed. The probability of intense rains can be characterized thought the returning period, obtained from the inverse of the accumulated frequency.

If a specific occurrence is fixed the curves relating intensity and duration are then known as Average Maximum Intensity curves. These curves can define from a real rain event until a simulated rain event for a particular returning period.

When approaching the IDF curves it has been detected that there is not a lot of valuable public information available. Therefore in this case the maps M5-60 minutes and M100-6 minutes were used.

United Kingdom is divided in 10 hydrological areas as it is shown below:

Figure 2. Hydrological regions for United Kingdom. Source: UKWIR

The catchment of study is located in the Kilmarnock's urban area and surroundings, hence we can define the area of study as the one located in section 2, which belongs to Scotland.

2.5.1. Rainfall 10 years returning period

Once defined the area of study FSR maps (Flow Studies Report) are checked. M5-60 map is presented under these lines:

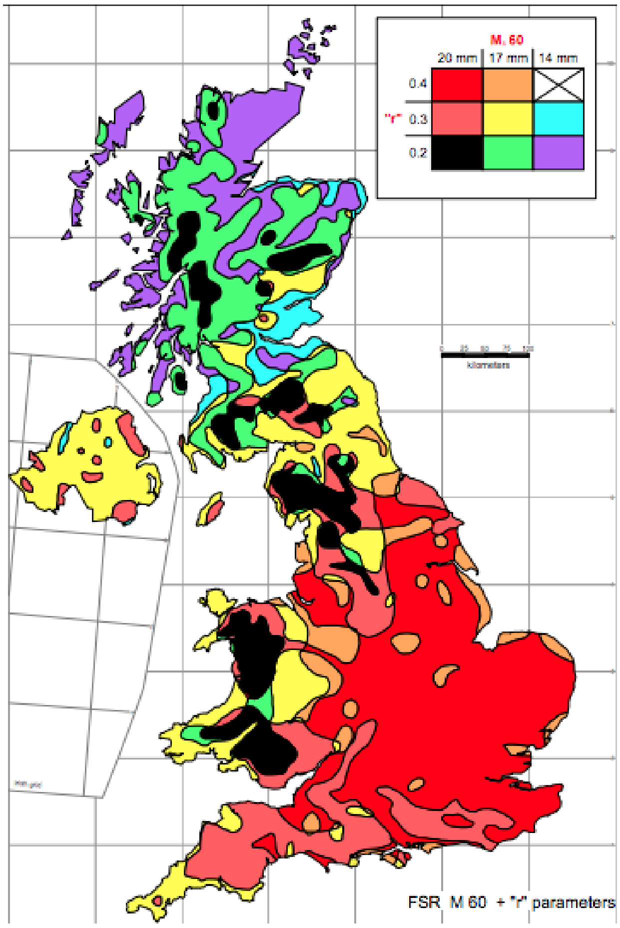

Figure 3. 5 year 60 minutes rainfall depth parameters United Kingdom. Source: Centre for Ecology and Hydrology

As pointed on the map, it can be seen that the area of study for this case corresponds to the green area. According to the legend, this area matches with a M5-60 of 17mm and a parameter "r" equal to 0.2. Moreover parameters Z1 and Z2, which depends on the impermeability of the ground and on the growth curve factor, are defined as 0.5 and 1.35 respectively. It is considered in this case that the ARF parameter (Area Reduction Factor) has a value of 1.

Lastly, once all the data is collected, calculations are done:

$$Rainfall\ Intensity = \frac{\frac{rainfall\ [mm]}{r} * Z1 * Z2 * ARF}{number\ of\ hours}$$

$$Rainfall\ Intensity = \frac{\frac{17}{0.2} * 0.35 * 1.35 * 1}{1} = 40.16 mm/h$$

When the rainfall intensity if obtained the time site manager table is filled and thus the expected rainfall for the returning periods of study is defined.

2.5.2. Rainfall 100 years returning period

For the same calculations and a returning period of 100 years of a 6 hours rainfall, the FSR maps give directly the amount of rainfall expected as shown in the following map:

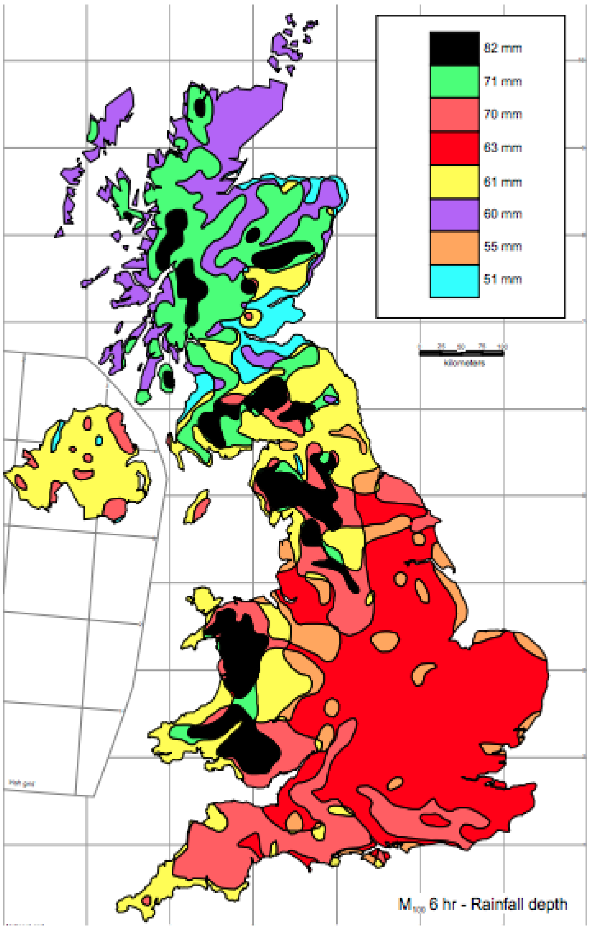

Figure 4. 100 years 60 minutes rainfall depth in United Kingdom. Source: Centre for Ecology and Hydrology

Accordingly with the equation used above for the rainfall intensity for a returning period of 5 years, for a 100 years returning period we obtain that:

$$Rainfall\ intensity = 71\ {}^{mm}/_{h}$$

2.5.3. Rainfall 500years returning period

Once obtained the maximum dairy rainfall intensity for 10 and 100 yeas returning period, it is seen that surprisingly there is no historical rainfall information available in Scotland's official sources which is need for the calculation of the rainfall for 500 years returning period.

In order to calculate the maximum daily rainfall intensity in this case, Gumbel distribution is used. For that in first instance basic statistic parameters such as arithmetic mean, typical deviation, number of data of the sample and "m" and "s" must be defined.

Number of data of the sample:

$$X_1 = 40.16\,mm/h$$

$$X_2 = 71mm/h$$

Arithmetic mean:

$$X_n = \sqrt{(X_1 - X_n)^2 + (X_2 - X_n)^2} = \sqrt{(40.16 - 55.58)^2 + (71 - 55.58)^2} = 21.8\,mm/h$$

Coefficients depending on the size of the sample:

$$\sigma_y = 0.4043$$

$$\mu_y = 0.4984$$

Probability of no overcoming the sample for 500 years returning period:

$$F(x) = 1 - \frac{1}{x} = 1 - \frac{1}{500} = 0.998$$

From these, it is obtained:

$$\beta = \frac{Typical\ deviation}{\mu_y} = 43.73$$

$$\mu = X_n - \left(\sigma_y * \beta\right) = 55.58 - (0.4043 * 43.73) = 37.894$$

Finally we obtain the not overcoming value as:

$$Maximum\ rainfall\ intensity\ in\ 1h\ for\ 500\ years\ returning\ period$$
$$= -\ln\left(-\ln\left(F(x)\right)\right) * \beta * \mu = -\ln(-\ln(0.998)) * 43.73 * 37.894 = 239.1\,mm/h$$

Once obtained the maximum average rainfall intensity of the day for all the returning periods, and also obtained the maximum daily rainfall intensity, it is calculated the average intensity for a "t" period of rainfalls using Temez equation.

$$I_t = I_{24} * (\frac{I_1}{I_{24}})^{\frac{28^{0.1}-t^{0.1}}{28^{0.1}-1}}$$

Where:

I_{24}: Average daily intensity = $P_d/24$

I_1: Average intensity on the rainiest hour of the day

t: Returning period

I_t: Average intensity during the period t

Returning Period (years)	Average daily intensity (mm/h)	Storm time (h)	Average intensity on the rainiest hour of the day (mm/h)	Parameter coefficient	Average intensity during the period t (mm/h)
10	5	24	40,16	8,032	5,595
100	8,75	24	71	8,114285714	9,796
500	28,5	24	239,1	8,389473684	31,967

Table 2. Rainfall intensity definition for using Temez equation

As seen on the table the parameter coefficient has a value of approximately 8. If we compare this value with the one fixed by ACA (Agència Catalana de l'Aigua) in Galicia, which has an Atlantic weather like Scotland, we can see that it is nearly the same value because the value fixed by ACA is 8 so this is an indicative that the calculations are correct.

From the results of the table, it is obtained the value for an average rainfall intensity for 24h, this result will be used for obtaining the IDF curves and the corresponding hyetographs.

Once the commented calculations are done, the Rational Method is used in order to find the design hyetograph for 10, 100 and 500 years returning period. From that, the following results tables are presented:

HYETOGRAPH FOR 10 YEARS RETURNING PERIOD						
Storm duration (h)		24	1440	min		
Rainfall intensity (mm/h)		5,54				
Rainfall in 24h (mm)		132,96				
Time interval (min)		30				
Time interval (min)	**Intensity (mm/h)**	**Rainfall accumulated (mm)**	**Rainfall (mm)**	**Partial intensity (mm/h)**	**Alternate rainfall (mm)**	**Alternate partial intensity (mm)**
30	97,33	48,67	48,67	97,33	0,62	1,24
60	55,90	55,90	7,24	14,47	0,73	1,46
90	40,41	60,62	4,72	9,43	0,57	1,14
120	32,11	64,22	3,61	7,21	0,63	1,26
150	26,86	67,15	2,93	5,86	0,76	1,51
180	23,21	69,63	2,48	4,96	0,70	1,41
210	20,52	71,82	2,19	4,38	0,77	1,54
240	18,44	73,76	1,94	3,88	0,86	1,72
270	16,78	75,51	1,75	3,50	0,92	1,85
300	15,43	77,15	1,64	3,28	1,05	2,09
330	14,29	78,60	1,44	2,89	1,10	2,19
360	13,33	79,98	1,39	2,77	1,27	2,54
390	12,50	81,25	1,27	2,54	1,44	2,89
420	11,79	82,53	1,28	2,56	1,75	3,50
450	11,15	83,63	1,10	2,19	2,19	4,38
480	10,59	84,72	1,10	2,19	2,93	5,86
510	10,09	85,77	1,05	2,09	4,72	9,43
540	9,64	86,76	1,00	1,99	48,67	97,33
570	9,23	87,69	0,92	1,85	7,24	14,47
600	8,86	88,60	0,91	1,83	3,61	7,21
630	8,52	89,46	0,86	1,72	2,48	4,96
660	8,21	90,31	0,85	1,70	1,94	3,88
690	7,92	91,08	0,77	1,54	1,64	3,28
720	7,66	91,92	0,84	1,68	1,39	2,77
750	7,41	92,63	0,70	1,41	1,28	2,56
780	7,18	93,34	0,71	1,43	1,10	2,19
810	6,97	94,10	0,76	1,51	1,00	1,99
840	6,77	94,78	0,68	1,37	0,91	1,83
870	6,58	95,41	0,63	1,26	0,85	1,70
900	6,41	96,15	0,74	1,48	0,84	1,68
930	6,24	96,72	0,57	1,14	0,71	1,43
960	6,08	97,28	0,56	1,12	0,68	1,37
990	5,94	98,01	0,73	1,46	0,74	1,48
1020	5,79	98,43	0,42	0,84	0,56	1,12
1050	5,66	99,05	0,62	1,24	0,42	0,84
1080	5,54	99,72	0,67	1,34	0,67	1,34

Table 3. Rainfall data for 10 years returning period

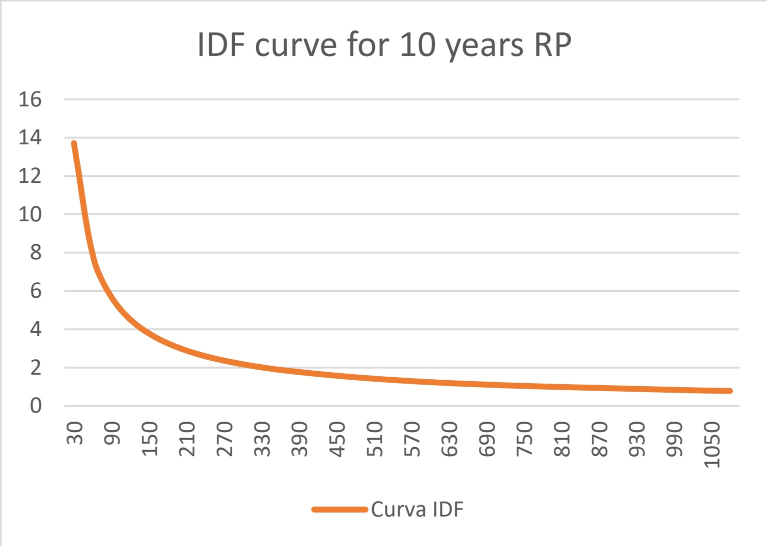

Figure 5. IDF curve for 10 years RP obtained from table 3

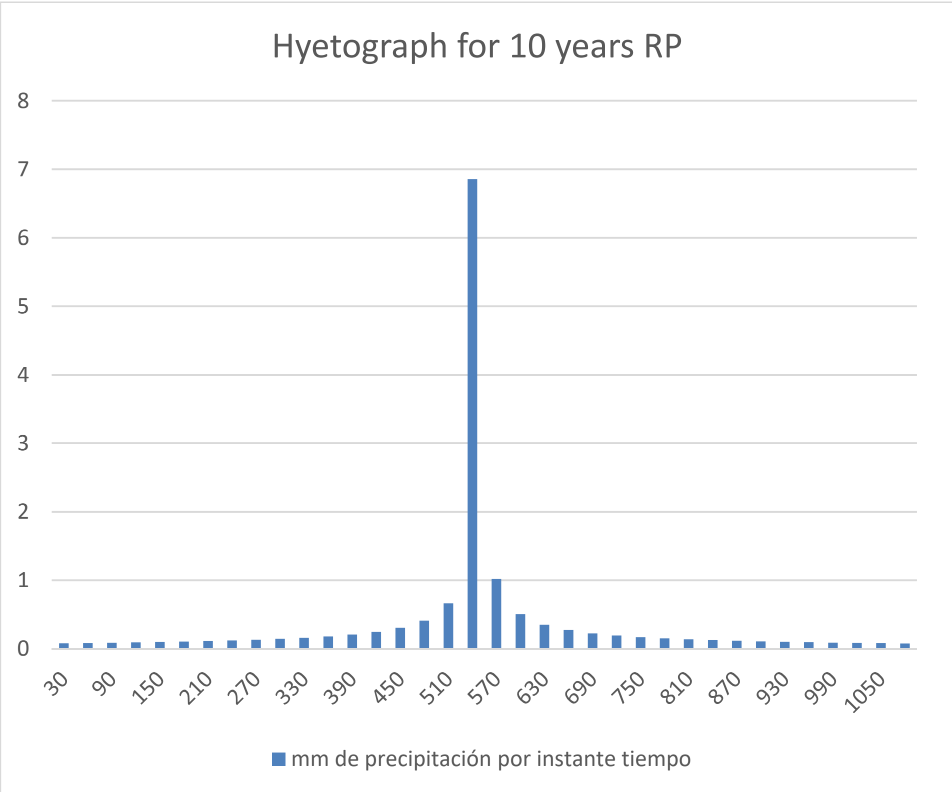

Figure 6. Hyetograph for 10 years RP obtained from table 3

HYETOGRAPH FOR 100 YEARS RETURNING PERIOD						
Storm duration (h)		24	1440	min		
Rainfall intensity (mm/h)		9,79				
Rainfall in 24h (mm)		234,96				
Time interval (min)		30				
Inteval (min)	**Intensity (mm/h)**	**Accumulated rainfall (mm)**	**Rainfall (mm)**	**Partial intensity (mm/h)**	**Alternate rainfall (mm)**	**Partial alternate intensity (mm)**
30	171,88	85,94	85,94	171,88	1,09	2,18
60	98,72	98,72	12,78	25,56	1,08	2,16
90	71,37	107,06	8,34	16,67	1,16	2,32
120	56,70	113,40	6,35	12,69	1,19	2,38
150	47,43	118,58	5,18	10,35	1,35	2,69
180	40,99	122,97	4,40	8,79	1,39	2,77
210	36,24	126,84	3,87	7,74	1,39	2,77
240	32,57	130,28	3,44	6,88	1,53	3,05
270	29,64	133,38	3,10	6,20	1,67	3,34
300	27,24	136,20	2,82	5,64	1,87	3,74
330	25,24	138,82	2,62	5,24	2,08	4,16
360	23,54	141,24	2,42	4,84	2,28	4,56
390	22,08	143,52	2,28	4,56	2,62	5,24
420	20,81	145,67	2,15	4,30	3,10	6,20
450	19,70	147,75	2,08	4,16	3,87	7,74
480	18,70	149,60	1,85	3,70	5,18	10,35
510	17,82	151,47	1,87	3,74	8,34	16,67
540	17,02	153,18	1,71	3,42	85,94	171,88
570	16,30	154,85	1,67	3,34	12,78	25,56
600	15,65	156,50	1,65	3,30	6,35	12,69
630	15,05	158,03	1,53	3,05	4,40	8,79
660	14,50	159,50	1,47	2,95	3,44	6,88
690	13,99	160,89	1,39	2,77	2,82	5,64
720	13,52	162,24	1,35	2,71	2,42	4,84
750	13,09	163,63	1,39	2,77	2,15	4,30
780	12,68	164,84	1,22	2,43	1,85	3,70
810	12,31	166,19	1,35	2,69	1,71	3,42
840	11,95	167,30	1,12	2,23	1,65	3,30
870	11,62	168,49	1,19	2,38	1,47	2,95
900	11,31	169,65	1,16	2,32	1,35	2,71
930	11,02	170,81	1,16	2,32	1,22	2,43
960	10,74	171,84	1,03	2,06	1,12	2,23
990	10,48	172,92	1,08	2,16	1,16	2,32
1020	10,23	173,91	0,99	1,98	1,03	2,06
1050	10,00	175,00	1,09	2,18	0,99	1,98
1080	9,78	176,04	1,04	2,08	1,04	2,08

Table 4. Rainfall data for 100 years returning period

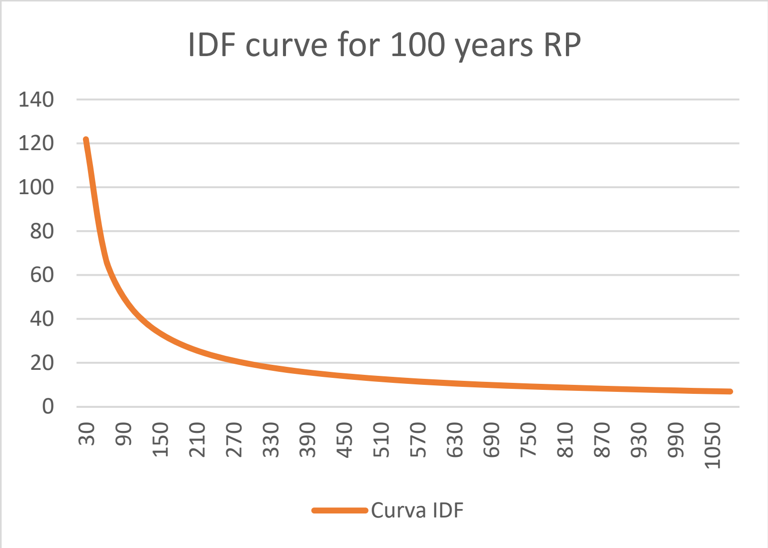

Figure 7. IDF curve obtained for 100 years RP obtained from table 4

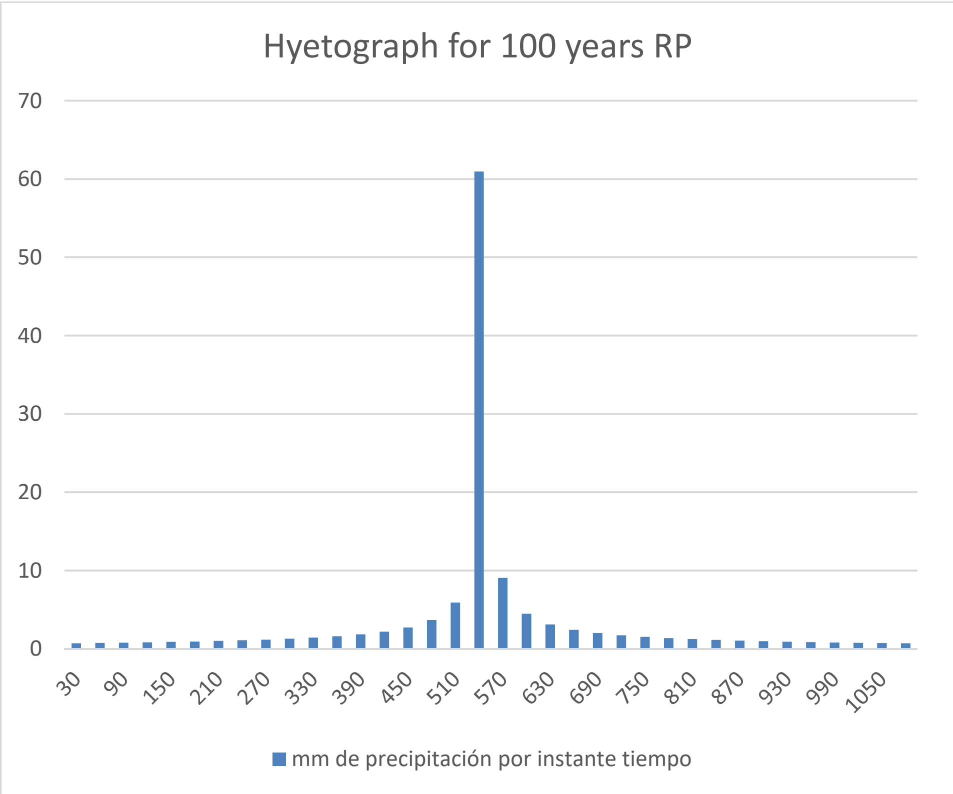

Figure 8. Hyetograph for 100 years RP obtained from table 4

HYETOGRAPH FOR 500 YEARS RETURNING PERIOD						
Storm duration (h)		24	1440	min		
Rainfall intensity (mm/h)		31,93				
Rainfall in 24h (mm)		766,32				
Time interval (min)		30				
Interval (min)	Intensity (mm/h)	Accumulated rainfall (mm)	Rainfall (mm)	Partial intensity (mm/h)	Alternate rainfall (mm)	Partial alternate intensity (mm)
30	561,42	280,71	280,71	561,42	3,24	6,48
60	322,45	322,45	41,74	83,48	3,52	7,04
90	233,13	349,70	27,25	54,49	3,60	7,19
120	185,00	370,00	20,31	40,61	3,86	7,72
150	154,92	387,30	17,30	34,60	4,11	8,22
180	133,90	401,70	14,40	28,80	4,33	8,67
210	118,36	414,26	12,56	25,12	4,70	9,40
240	106,37	425,48	11,22	22,44	4,98	9,95
270	96,80	435,60	10,12	20,24	5,48	10,95
300	88,98	444,90	9,30	18,60	5,98	11,96
330	82,45	453,48	8,58	17,15	6,62	13,23
360	76,90	461,40	7,93	15,85	7,44	14,89
390	72,13	468,85	7,44	14,89	8,58	17,15
420	67,98	475,86	7,02	14,03	10,12	20,24
450	64,33	482,48	6,62	13,23	12,56	25,12
480	61,09	488,72	6,25	12,49	17,30	34,60
510	58,20	494,70	5,98	11,96	27,25	54,49
540	55,60	500,40	5,70	11,40	280,71	561,42
570	53,25	505,88	5,48	10,95	41,74	83,48
600	51,11	511,10	5,23	10,45	20,31	40,61
630	49,15	516,08	4,98	9,95	14,40	28,80
660	47,35	520,85	4,77	9,55	11,22	22,44
690	45,70	525,55	4,70	9,40	9,30	18,60
720	44,17	530,04	4,49	8,98	7,93	15,85
750	42,75	534,38	4,33	8,67	7,02	14,03
780	41,43	538,59	4,22	8,43	6,25	12,49
810	40,20	542,70	4,11	8,22	5,70	11,40
840	39,04	546,56	3,86	7,72	5,23	10,45
870	37,96	550,42	3,86	7,72	4,77	9,55
900	36,95	554,25	3,83	7,66	4,49	8,98
930	35,99	557,85	3,60	7,19	4,22	8,43
960	35,09	561,44	3,60	7,19	3,86	7,72
990	34,24	564,96	3,52	7,04	3,83	7,66
1020	33,43	568,31	3,35	6,70	3,60	7,19
1050	32,66	571,55	3,24	6,48	3,35	6,70
1080	31,93	574,74	3,19	6,38	3,19	6,38

Table 5. Rainfall data for 500 years returning period

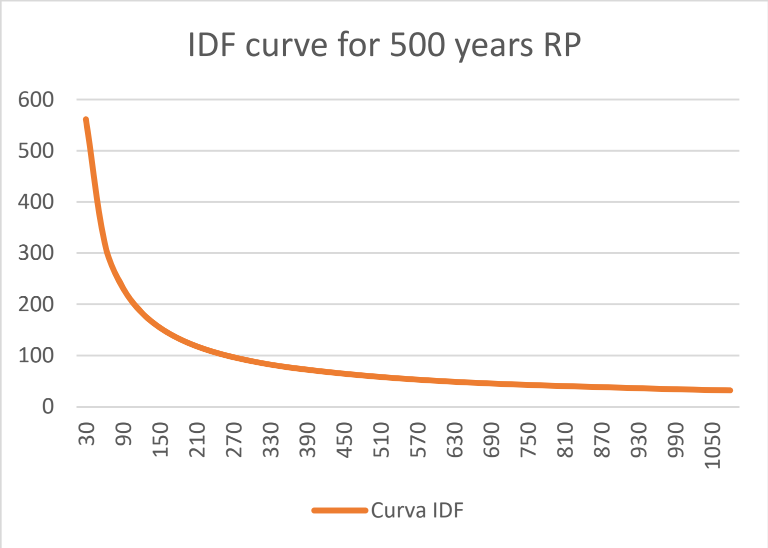

Figure 9. IDF curve for 500 years RP obtained from table 5

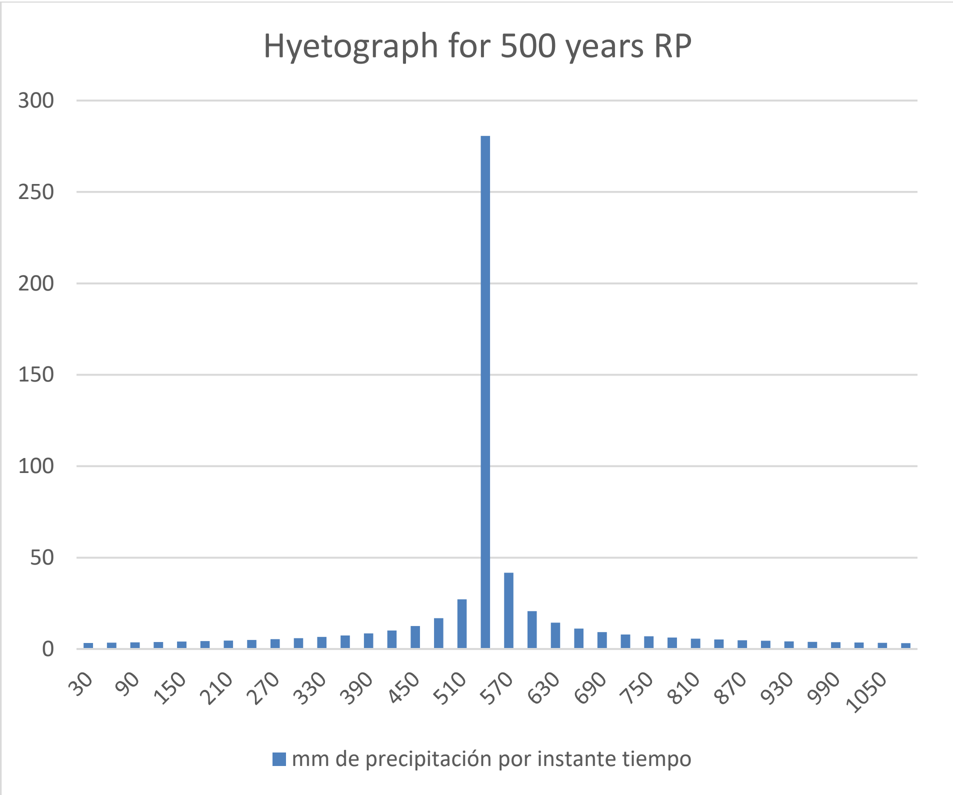

Figure 10. Hyetograph for 500 years RP obtained from table 5

2.6. Control section

In this section the start and finish intervals are defined as well as the starting and finishing hours. These parameters will designate the limits of the outflow hydrograph and consequently the results. The designation of the parameters are presented underneath for each returning period:

10 years returning period

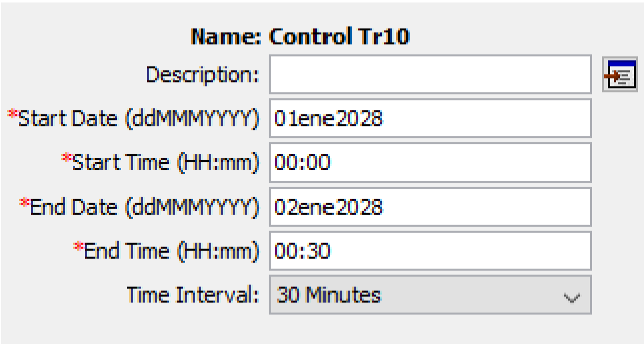

Figure 11. HEC-HMS Control section for 10 years returning period

100 years returning period

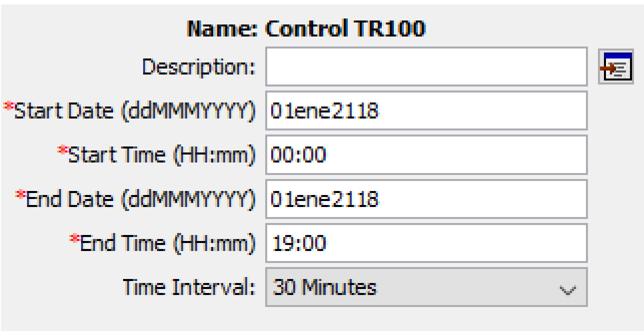

Figure 12. HEC-HMS Control section for 100 years returning period

500 years returning period

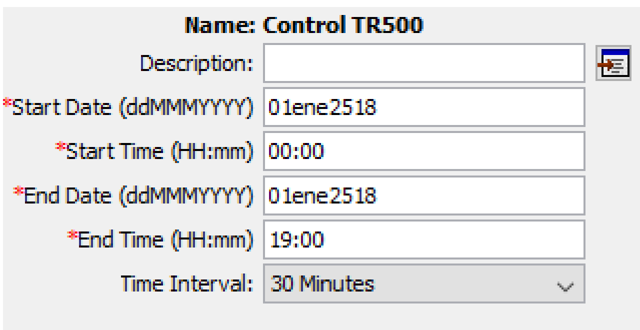

Figure 13. HEC-HMS Control section for 500 years returning period

2.7. Surface runoff

The surface runoff is the portion of rainfall that is not evaporated. Is a parameter that allows calculating the net rainfall from a determined rain. This parameter includes the interception by vegetation, storage in small low-lying land and infiltration.

Since in this study the program HEC-HMS is used, the Losses Method is calculated using the SCS Curve Number (SCS-CN) model. In this, the Curve Number is the only parameter to take into consideration. The Curve Number is directly related to the type of soil topographic information and land uses. The reason for using this method is that for other methods, such as Rational Method, some calibrated parameters that are not currently available in this case are required.

Consequently and having the topography, using the following tables each subbasin curve number is obtained:

Land Use Description on Input Screen	**Description and Curve Numbers from TR-55**					
	Cover Description		**Curve Number for Hydrologic Soil Group**			
	Cover Type and Hydrologic Condition	% Impervious Areas	A	B	C	D
Agricultural	Row Crops - Staight Rows + Crop Residue Cover- Good Condition (1)		64	75	82	85
Commercial	Urban Districts: Commerical and Business	85	89	92	94	95
Forest	Woods(2) - Good Condition		30	55	70	77
Grass/Pasture	Pasture, Grassland, or Range(3) - Good Condition		39	61	74	80
High Density Residential	Residential districts by average lot size: 1/8 acre or less	65	77	85	90	92
Industrial	Urban district: Industrial	72	81	88	91	93
Low Density Residential	Residential districts by average lot size: 1/2 acre lot	25	54	70	80	85

Open Spaces	Open Space (lawns, parks, golf courses, cemeteries, etc.)(4) Fair Condition (grass cover 50% to 70%)		49	69	79	84
Parking and Paved Spaces	Impervious areas: Paved parking lots, roofs, drivesways, etc. (excluding right-of-way)	100	98	98	98	98
Residential 1/8 acre	Residential districts by average lot size: 1/8 acre or less	65	77	85	90	92
Residential 1/4 acre	Residential districts by average lot size: 1/4 acre	38	61	75	83	87
Residential 1/3 acre	Residential districts by average lot size: 1/3 acre	30	57	72	81	86
Residential 1/2 acre	Residential districts by average lot size: 1/2 acre	25	54	70	80	85
Residential 1 acre	Residential districts by average lot size: 1 acre	20	51	68	79	84
Residential 2 acres	Residential districts by average lot size: 2 acre	12	46	65	77	82
Water/ Wetlands		0	0	0	0	0

Table 6. Curve numbers for hydrologic soil groups. Source: USDA

Once each subbasin is defined the following equation is used:

$$CN = \sum \frac{CN_i}{100} * \%\ of\ the\ subbasin$$

2.7.1. *Irvine subbasin*

Land Use	Hydraulic Condition	Curve Number	% of the subbasin
Urban District: High density	B	41	15
Urban district: Low Density	B	41	10
Urban district: Industrial	B	37	2,5
Grass/Pasturate	B	71	46,5
Forest	A	58	26

Table 7. SCS Curve number definition for Irvine

$$CN_{Irvine} = \left(\frac{41 * 15}{100}\right) + \left(\frac{41 * 10}{100}\right) + \left(\frac{37 * 2.5}{100}\right) + \left(\frac{71 * 46.5}{100}\right) + \left(\frac{58 * 26}{100}\right) = 59.27$$

2.7.2. *Kilmarnock subbasin*

Land Use	Hydraulic Condition	Curve Number	% of the basin
Urban District: High density	B	41	80
Urban district: Low Density	B	41	5
Urban district: Industrial	B	37	7,5
Grass/Pasturate	B	71	5
Forest	A	58	2,5

Table 8. SCS Curve number definition for Kilmarnock

$$CN_{Kilmarnock} = 42.625$$

2.7.3. Ayrshire subbasin

Since Ayrshire River has two tributaries, a topographic distinction of each effluent must be done. So a curve number for the west section and a curve number for the east section are calculated and subsequently the final curve number is obtained through the average of the results.

For the west section:

Land Use	Hydraulic Condition	Curve Number	% of the basin
Urban District: High density	B	41	0
Urban district: Low Density	B	41	5
Urban district: Industrial	B	56	0
Grass/Pasturate	B	71	95
Forest	A	58	0

Table 9. SCS Curve number definition for west Ayrshire

$$CN_{west} = 69.5$$

For the east section:

Land Use	Hydraulic Condition	Curve Number	% of the basin
Urban District: High density	B	41	0
Urban district: Low Density	B	41	10
Urban district: Industrial	B	56	15
Grass/Pasturate	B	71	75
Forest	A	58	0

Table 10. SCS Curve number definition for east Ayrshire

$$CN_{east} = 65.75$$

And making the average of the results above:

$$CN_{Ayrshire} = \left(\frac{69.5 + 65.75}{2}\right) = 67.625$$

2.8. Results

2.8.1. For 10 years returning period

Once is done the interpolation for the rainfall intensity considering a returning period of 100 years and an average rainfall intensity of 40.16mm/h, it is proceed to calculate the maximum flood. For that, a maximum rainfall day was simulated.

Project: HEC-HMS def   Simulation Run: Run TR 10

Start of Run: 01ene2028, 00:00   Basin Model: Study Basin
End of Run: 02ene2028, 00:30   Meteorologic Model: Precipitación Glasgow TR 10
Compute Time:DATA CHANGED, RECOMPUTE   Control Specifications:Control Tr 10

Show Elements: All Elements   Volume Units: MM 1000 M3   Sorting: Hydrologic

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (MM)
Irvine River	17.00000	104.9	01ene2028, 09:30	37.44
Ayrshire Water	0.79704	3.2	01ene2028, 09:30	31.10
Irvine Rivesr + Tributary	17.79704	108.0	01ene2028, 09:30	37.15
Kilmarnock Water	14.00000	36.7	01ene2028, 09:30	18.99
Sumpropagate2	17.79704	87.1	01ene2028, 10:00	37.06
Junction Kilmarnock-Irvine	31.79704	107.8	01ene2028, 10:00	29.10
Sumpropagate1	31.79704	94.6	01ene2028, 10:30	29.00
study Output	31.79704	94.6	01ene2028, 10:30	29.00

Figure 14. HEC-HMS Results for 10 years returning period

As seen on the table, the maximum flood is happening on the junction between the water coming from river Irvine and the water coming from the tributaries in addition with the water from Kilmarnock's urban area.

This peak is happening at 10:00h with a consequent peak flood of 107.8m^3/s. The value is the sum of the peaks of "Sumpropagate2" hydrograph plus Kilmarnock water's hydrograph. This is clearly represented in the following graph:

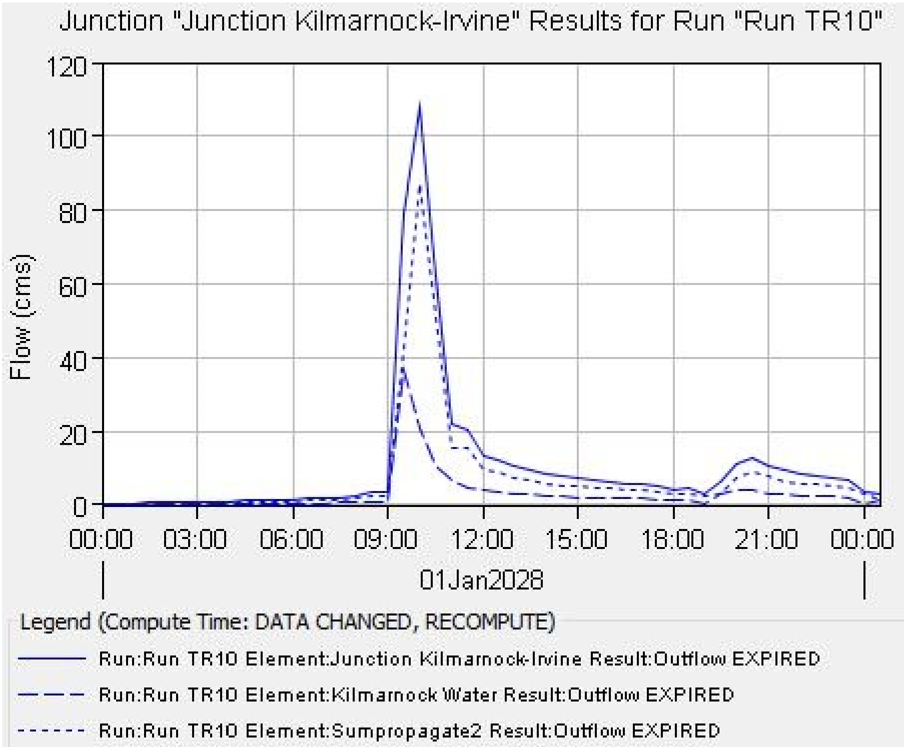

Figure 15. HEC-HMS Representation of the partial hydrographs and its sum for 10 years returning period

As seen in the representation above there is two inflow hydrographs, its sum has as a result the total hydrograph. In this last can be clearly seen an ascending zone from 9:00h until 10:00h, time when the flood peak is reached, in other words, the peak flow rate. Subsequently a recession curve is produced until reaching the base flow rate and, once entering in this area it is known as depletion curve.

In the hydrograph it is also seen a base runoff at 20m^3/s, this runoff is due to initial infiltration losses and to the soil's permeability.

The hyetograph is the graphical representation of the relationship between the rainfall intensity and time. This chart is very useful in representing the characteristics of a storm since the area under the hyetograph gives the total rainfall occurred in the represented period, and for that reason is of particularly interest when developing the design storm to predict extreme flood events.

The corresponding hyetograph for 10 years returning period shows that in the storm peak the rainfall intensity can accumulate 41.6mm/h during a rainfall period 1h as it is seen below:

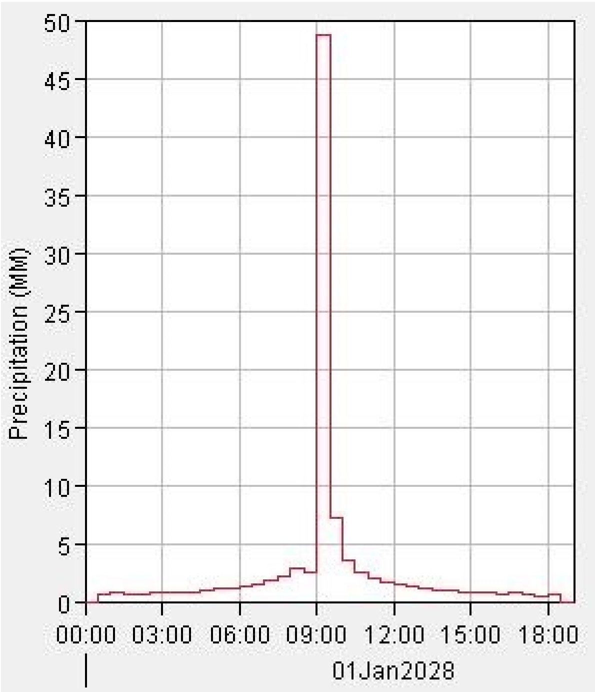

Figure 16. HEC-HMS Hyetograph for 10 years returning period

The hyetograph results calculated using HEC-HMS are very similar to the ones represented in figure 6 calculated using the rainfall data, which indicates that our initial results were pretty accurate.

2.8.2. For 100 years returning period

On the same way than for the calculation of the maximum flood for a 10 years returning period, before doing the calculation of the probable maximum flood it has been done the rainfall interpolation for a 100 years returning period with a rainfall intensity of 71mm/h. As done in the case before, a simulation of a maximum intensity rainfall day is done.

Project: HEC-HMS def Simulation Run: Run TR 100

Start of Run: 01ene2118, 00:00
End of Run: 01ene2118, 19:00
Compute Time:DATA CHANGED, RECOMPUTE

Basin Model: Study Basin
Meteorologic Model: Precipitación Glasgow TR 10
Control Specifications:Control TR 100

Show Elements: All Elements Volume Units: MM 1000 M3 Sorting: Hydrologic

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (MM)
Irvine River	17.00000	309.4	01ene2118, 09:30	84.50
Ayrshire Water	0.79704	13.8	01ene2118, 09:30	80.85
Irvine Rivesr + Tributary	17.79704	323.2	01ene2118, 09:30	84.33
Kilmarnock Water	14.00000	142.2	01ene2118, 09:30	50.42
Sumpropagate2	17.79704	256.6	01ene2118, 10:00	83.91
Junction Kilmarnock-Irvine	31.79704	328.8	01ene2118, 10:00	69.17
Sumpropagate1	31.79704	296.0	01ene2118, 10:00	68.80
study Output	31.79704	296.0	01ene2118, 10:00	68.80

Figure 17. HEC-HMS Results for 100 years retuning period

Same than on the previous case, the maximum flow rate is located in the junction between river Irvine and its tributaries, and with the water coming from Kilmarnock's urban area.

The flood peak is produced at 10:00h with a value of 328.8mm/h. This value is the sum of the entrance inflow from the Kilmarnock's water plus the water coming from "Sumpropagate2". The indicated is the junction between river Irvine and its tributaries (Ayrshire Water).

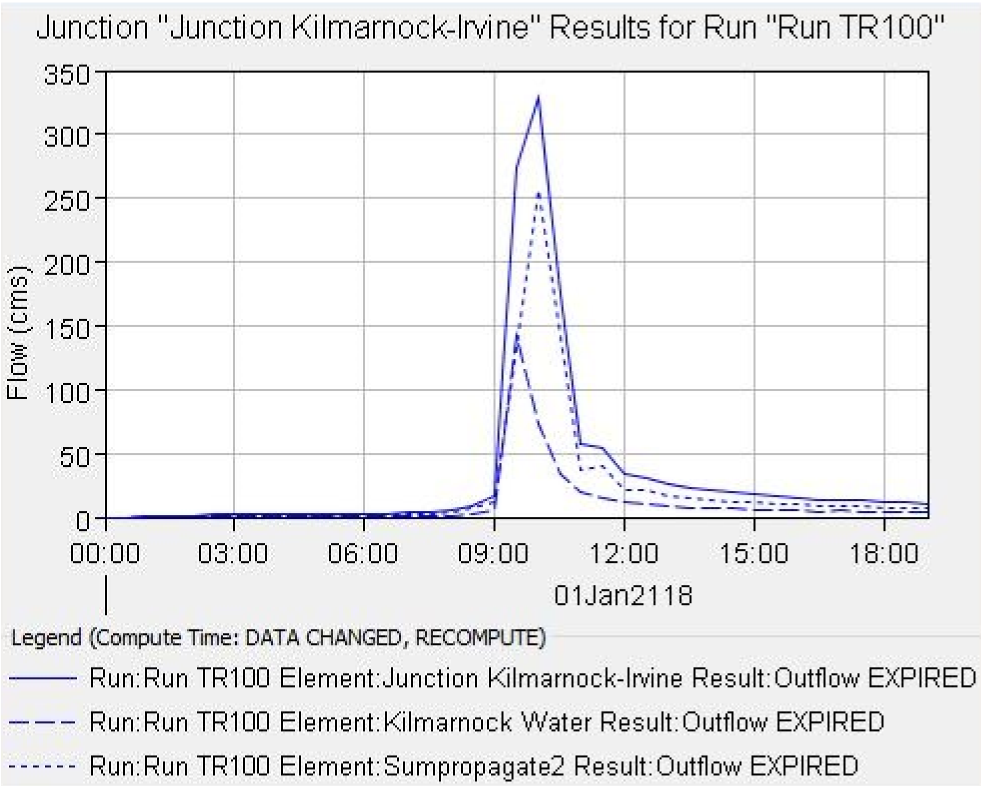

Figure 18. HEC-HMS Representation of the partial hydrographs and its sum for 100 years returning period

From 9:00h until 10:00h there is a clear ascendant zone where the peak flow rate is produced. Once the peak flow rate is surpassed it comes the recession curve until reaching the base flow rate, produced due to the base runoff. In this case the runoff value is as well 50m^3/s since this last does not depend on the rainfall intensity but it depends on the curve's number, the initial infiltration of the ground and its permeability.

For 100 years returning period it is seen on the hyetograph that rainfall intensity during the storm peak can accumulate71mm/h of water during a 1h period of time:

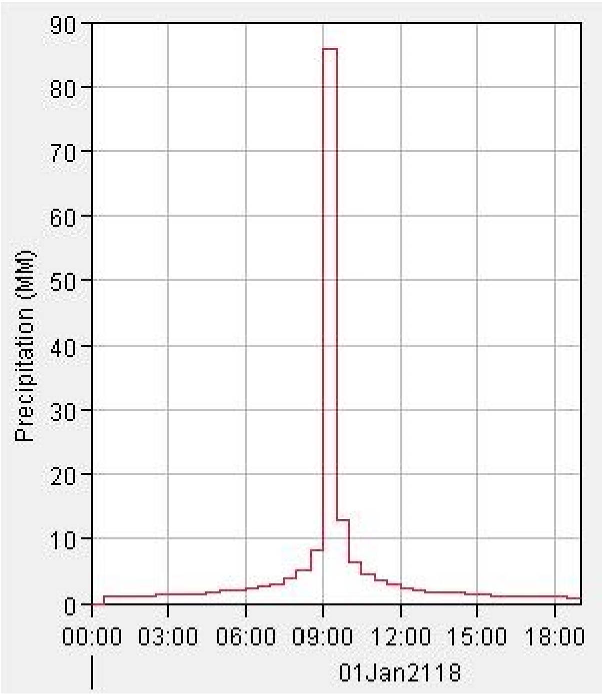

Figure 19. HEC-HMS Hyetograph for 100 years returning period

Same as in the previous case, the hyetograph obtained using the software and the one obtained with the rainfall data in figure 8 are very similar making the initial results trustable.

2.8.3. For 500 years returning period

With this theoretical value the software Hec-HMS can be executed and the following results are obtained:

Project: HEC-HMS def   Simulation Run: Run Tr 500

Start of Run: 01ene2518, 00:00   Basin Model: Study Basin
End of Run: 01ene2518, 19:00   Meteorologic Model: Precipitación Glasgow TR10
Compute Time: 13jun2018, 19:40:07   Control Specifications: Control TR500

Show Elements: All Elements   Volume Units: MM / 1000 M3   Sorting: Hydrologic

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (MM)
Irvine River	17.00000	322.2	01ene2518, 09:30	173.01
Ayrshire Water	0.79704	82.5	01ene2518, 09:30	446.57
Irvine Rivesr + Tributary	17.79704	404.7	01ene2518, 09:30	185.26
Kilmarnock Water	14.00000	139.7	01ene2518, 09:30	113.04
Sumpropagate2	17.79704	361.6	01ene2518, 10:00	183.93
Junction Kilmarnock-Irvine	31.79704	494.6	01ene2518, 10:00	152.72
Sumpropagate1	31.79704	461.2	01ene2518, 10:30	151.48
study Output	31.79704	461.2	01ene2518, 10:30	151.48

Figure 20. HEC-HMS Results for 500 years returning period

As seen in the results and accordingly to the previous calculations, the maximum flow rate is located in the junction between river Irvine and its tributaries, and with the water flowing from Kilmarnock area.

This peak is happening at 10:00h with a consequent peak flood of 494.6m^3/s. The value is the sum of the peaks of "Sumpropagate2" hydrograph plus Kilmarnock water's hydrograph. This is represented in the following graph:

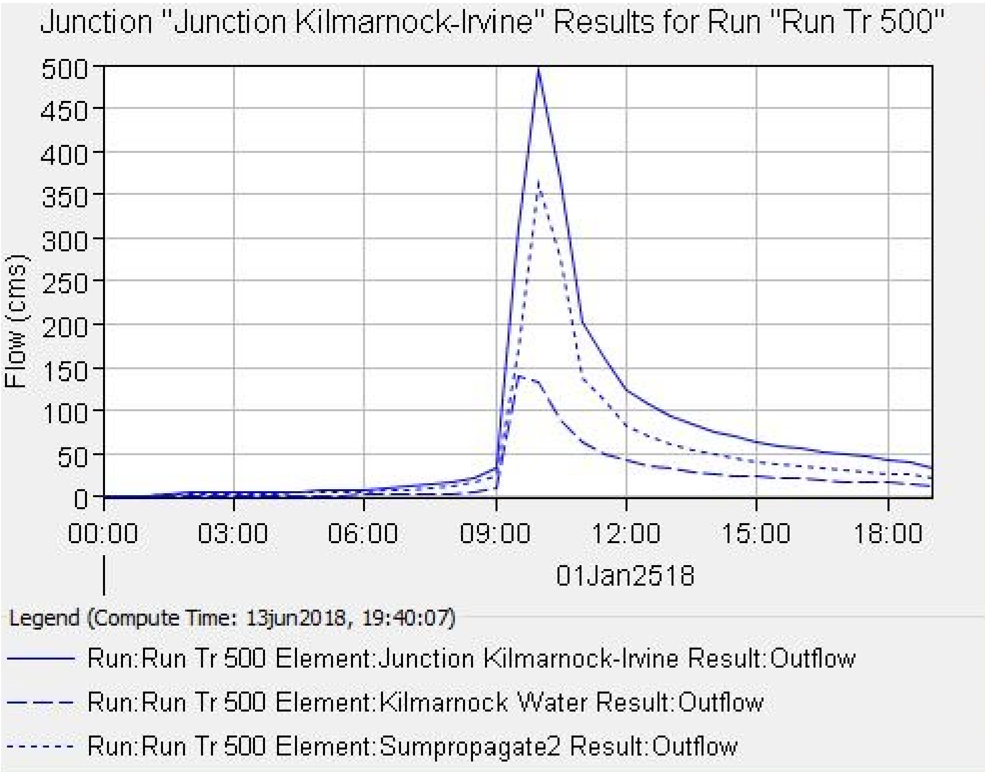

Figure 21. HEC-HMS Representation of the partial hygrograms and its sum for 500 years returning period

Finally for 500 years returning period and according to the corresponding hyetograph, during the storm peak the rainfall intensity accumulates 239.1mm/h during a 1h period of time as shown below:

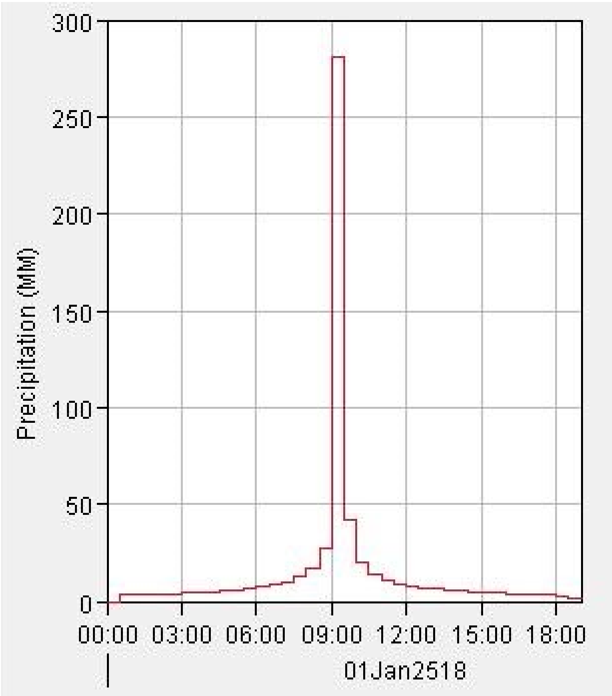

Figure 22. HEC-HMS Hyetograph for 500 years returning period

Finally the hyetograph obtained using HEC-HMS shows up a coincidence with the one represented in figure 10.

2.9. Basin calculations

Geometrical parameters

Area (S) = 33.146 Km^2
Maximum Length (L) = 12.95 km
Average slope (i) = 0.0830 m/m

Auxiliary data

Maximum elevation = 500 m
Minimum elevation = 0 m
Maximum change level= 0.5 Km

Lag Time

Urban degree (μ) = 0.0500 Km2/Km2

Maximum Concentration time Curve = 4.09h

Average Concentration Time Curve = 2.90h

Lag time Irvine River = 1.25h

Lag time Kilmarnock Water = 0.42h

Lag time Ayrshire Water = 0.17h

Surface runoff

Maximum Curve Number = 69.5 (West Ayrshire Water)

Minimum Curve Number = 49.625 (Kilmarnock Water)

Average Curve Number = 59.286

Coefficients calculation

K of Muskingum Irvine River = 2.15

K of Kilmarnock Water = 0.72

K of Ayrshire Water= 0.30

Resuming table

Returning period (years)	Rainfall Intensity (mm/h)	Irvine River Flow Rate (m^3/s)	Kilmanrock Water Flow Rate (m^3/s)	Maximum Flow Rate (m^3/s)
10	40.16	87.1	36.7	107.8
100	71	256.6	142.2	328.8
500	239.1	361.6	193.7	494.8

Table 11. Resuming table with maximum flow rates for different returning periods

3. Hydraulic calculations

3.1. Open channel flow calculations

The open channel flow calculations have been done with the software HEC-Ras developed by the Hydrologic Engineering Center of US Army Corps of Engineers from the United States of America for the calculation of the open channel flow profile in steady gradually varied flow in natural or construction channels.

The calculation procedure is based on solving the one-dimensional energy equation section-by-section from the boundary conditions using the Standard Step Method. In this method, knowing the sheet of water flow depth in one section, the one from the next section is assumed. With the hydraulic data of both sections (area, wet perimeter, hydraulic radio, …) the friction losses are calculated (using Manning) and the losses due to changing section. With the lasts, the open channel flow depth of the second section is related using the first section, and later it is compared with the assumed flow depth. Then the process is repeated until the error is acceptable.

The program allows irregular geometries with more roughness for each section. It also calculates the effect of obstruction such as bridges including the possibility of load-flow or spillway.

The version of HEC-RAS (5.0.3) used allows mixed flow regime (from subcritical to supercritical and backwards) and non-steady flow.

The results are represented by means of graphs or tables.

The program has the following limitations:

- Accepts the steady or non-steady flow.
- Gradually varied flow, excepting hydraulic structures such as bridges, side-channel spillways, dam gates, etc.
- One-dimensional flow
- Valid for channels or rivers with slope lower than 10%
- Valid for fixed riverbed

The velocity and flow depth at any channel section will be determined using Bernoulli's equation between two sections.

$$z_1 + y_1 + \frac{{v_1}^2}{2g} = z_2 + y_2 + \frac{{v_2}^2}{2g}$$

Where:

z_i: Riverbed depth (m)

y_i: Flow depth (m)

$\frac{{v_i}^2}{2g}$: Velocity height

Δh_2: Head losses

Cross section losses will be calculated using Manning's equation.

3.2. Modeling

In order the use the program under steady-flow conditions there is three main parameter that need to be defined: geometric data, flow data and hydraulic design.

3.2.1. Geometric data

In this part the riverbed cross section geometries are defined as well as the sides. It also needs to be defined the distance between the axis and the depth, the distance between sections, Manning's roughness for the riverbed and flowpaths, contraction and expansion coefficients (0.1 and 0.3 respectively) and any existent structure.

In order to have the most accurate representation of the river's behaviour, it is proceed to run the steady flow analysis for the 10, 100 and 500 years returning period flow rates already obtained and later, to add more cross-sections according to the results until they are satisfying enough.

The initial geometric data is represented below:

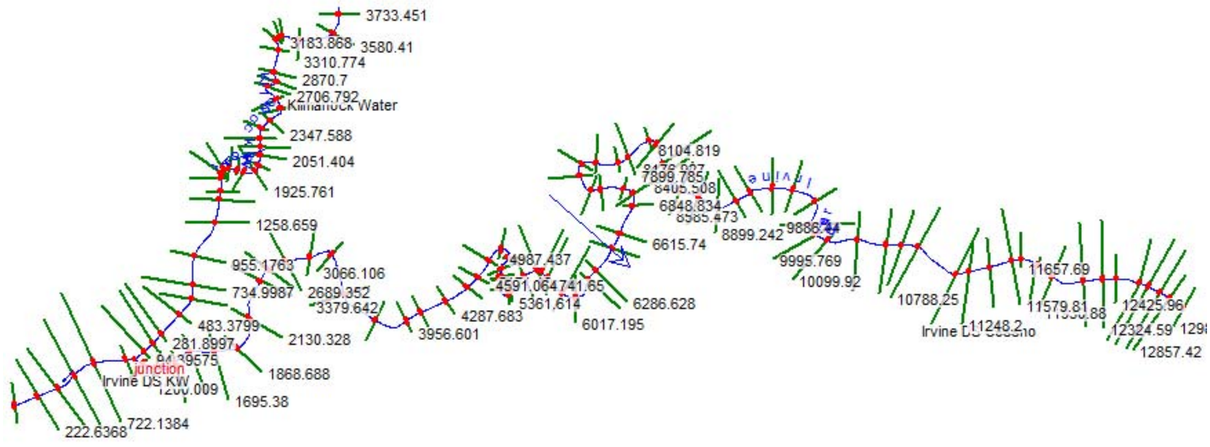

Figure 23. Initial geometric data

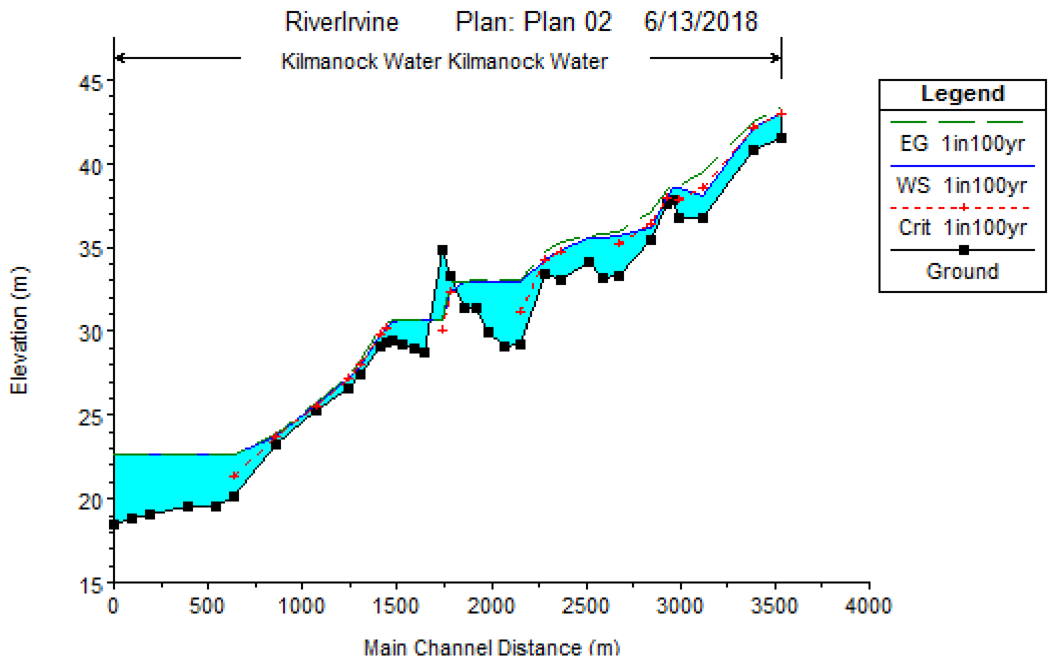

Figure 24. Initial profile for Kilmarnock Water

For Kilmarnock Water it is seen clearly that a considerable number of cross sections should be included. To add this new cross sections we create a new cross section, insert the coordinates, define LOB channel and ROB, and finally the station and the elevation are defined. All this information is provided by doing cross sections on the original topography of the area.

In this first attempt, cross sections will be added in base of the following observations of the profile:

- There is a big slope change between 1440 and 1750m, and 1750 and 2070m main channel distance.
- There is a considerably change in the river direction from 1440 to 3500m main channel distance. Abrupt changes on flow direction can change the flow regime so it is important to detail as much as possible.
- There is a urban area from 1300 to 3100m main channel distance so it is convenient to indicate with more cross sections the change of Manning's roughness.
- Existence of hydraulic structures.

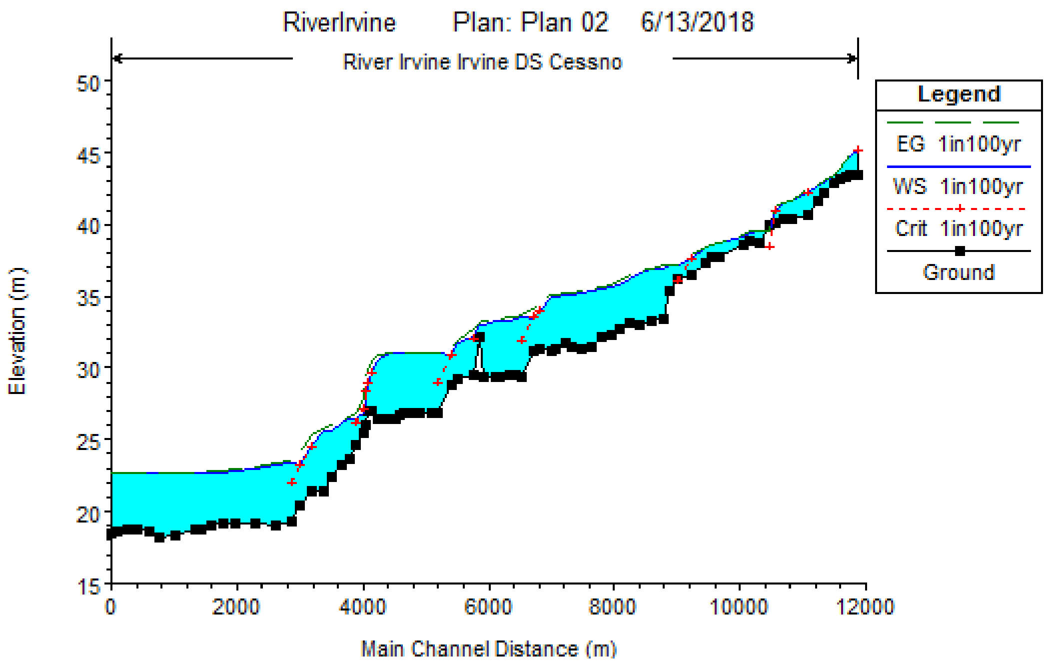

Figure 25. Initial profile for River Irvine DS Cessno

For River Irvine DS Cessno cross sections will be added in base of the following observations:

- There are considerably changes in the river direction from 3000 to 7000m main channel distance. Abrupt changes on flow direction can change the flow regime so it is important to detail as much as possible.
- Existence of hydraulic structures.

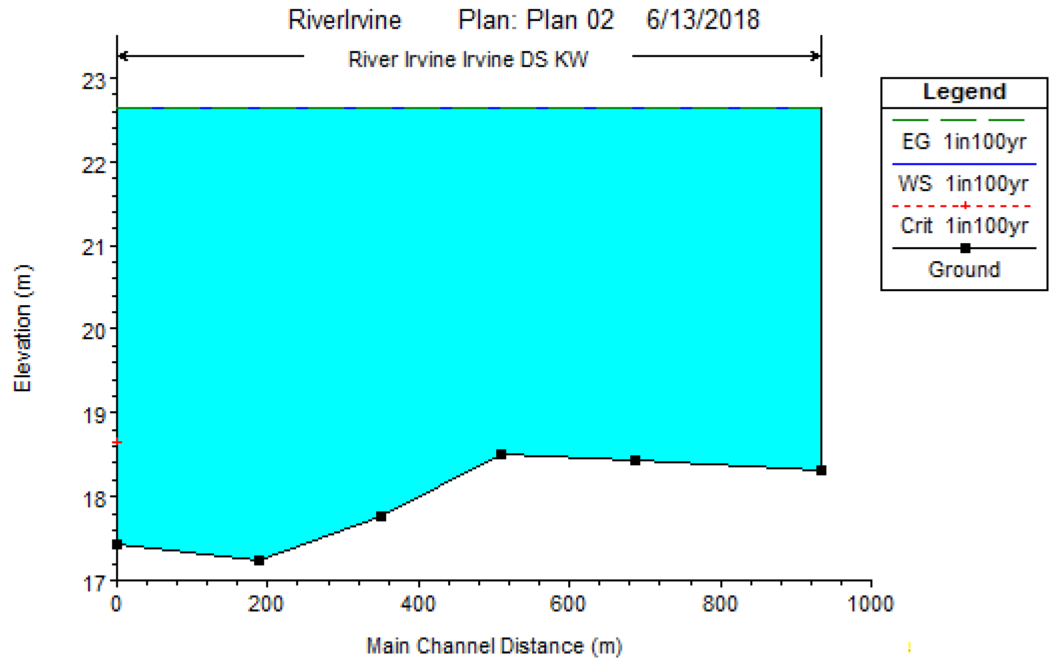

Figure 26. Initial profile for River Irvine DS KW

River Irvine DS KW is pretty good defined, some extra cross-sections will be added in the junction and along the longitude in order to increase the quality of the results.

After doing the already commented changes, the following geometry is obtained:

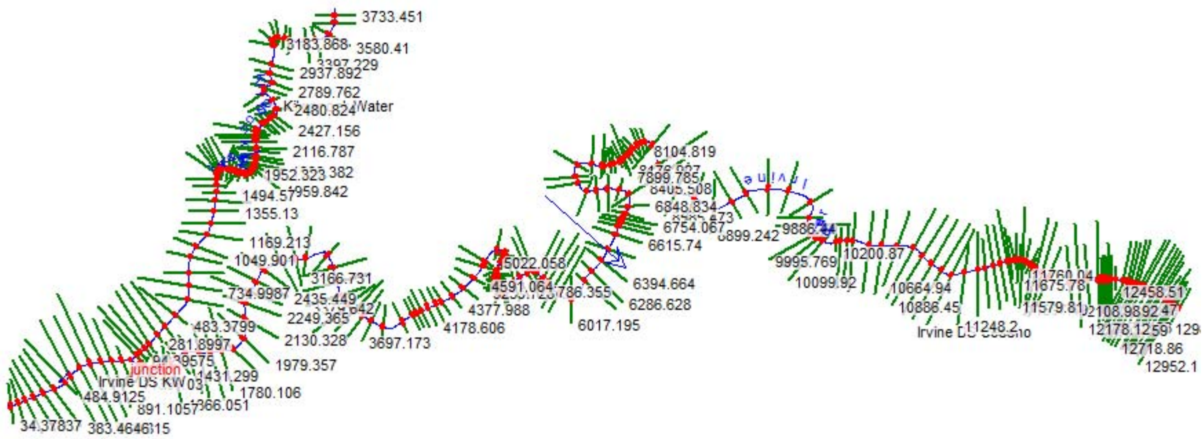

Figure 27. First attempt geometric data

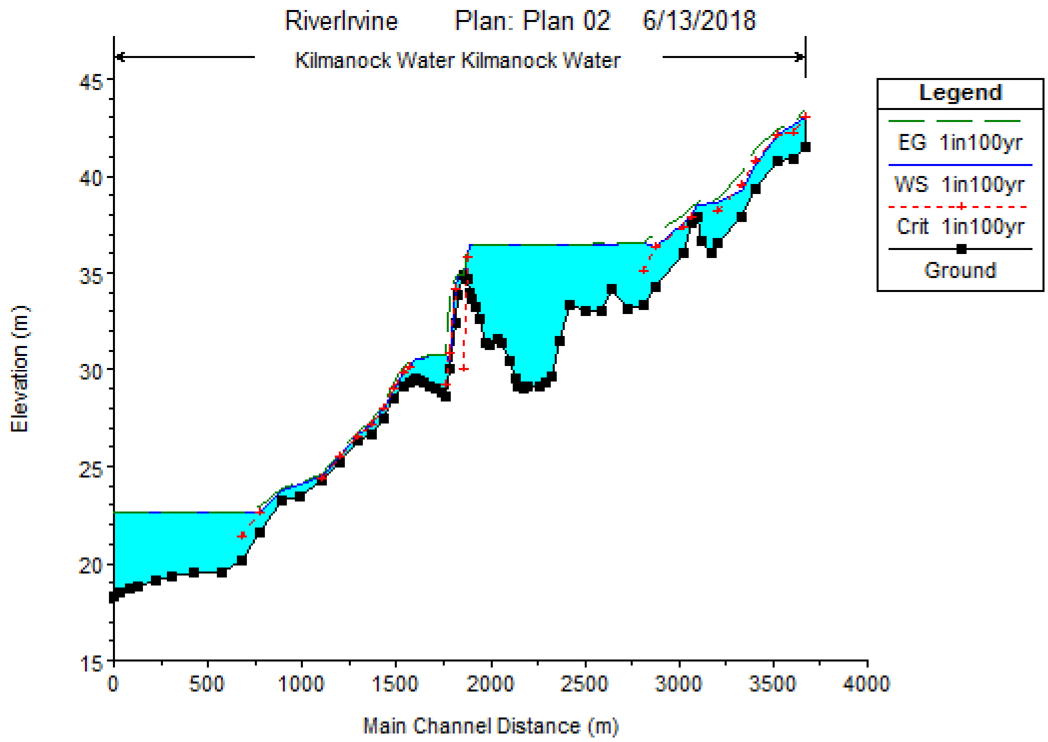

Figure 28. First attempt profile for Kilmarnock Water

A considerably change on the quality of the water profile of Kilmarnock Water can be appreciated after adding more cross sections. However, some extra cross sections should be defined from 1700 to 2100 to have the most accurate results of the flow behaviour.

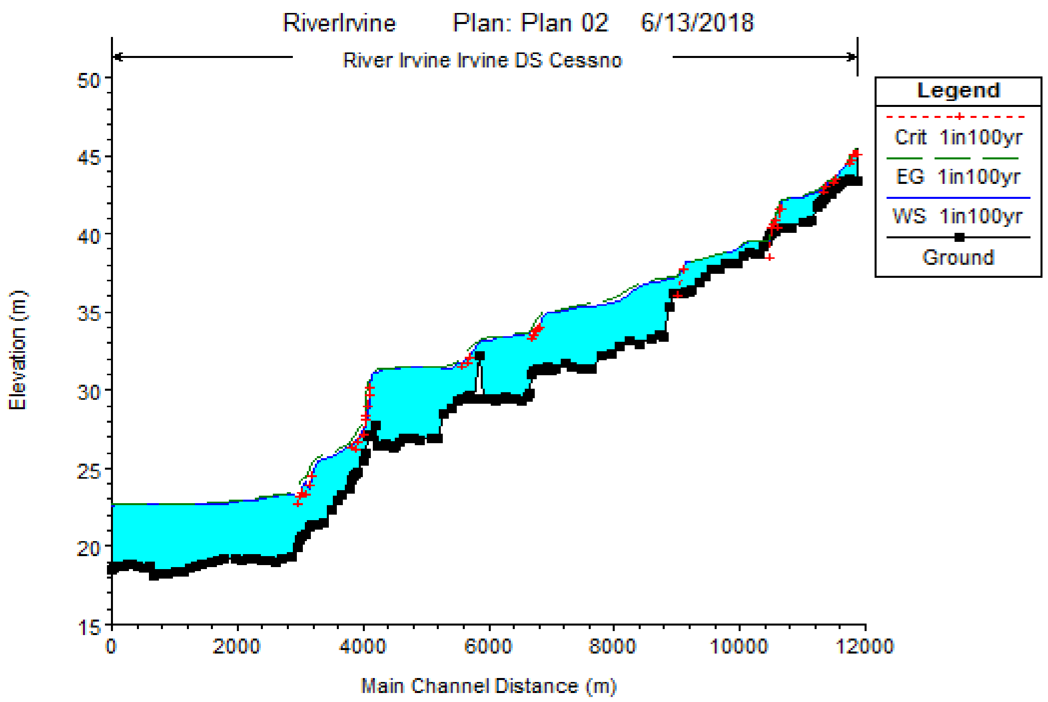

Figure 29. First attempt profile for River Irvine DS Cessno

After adding cross sections in the discussed areas of the river, a gain on the flow profile quality it is seen but without significant changes, so no more cross sections will be needed.

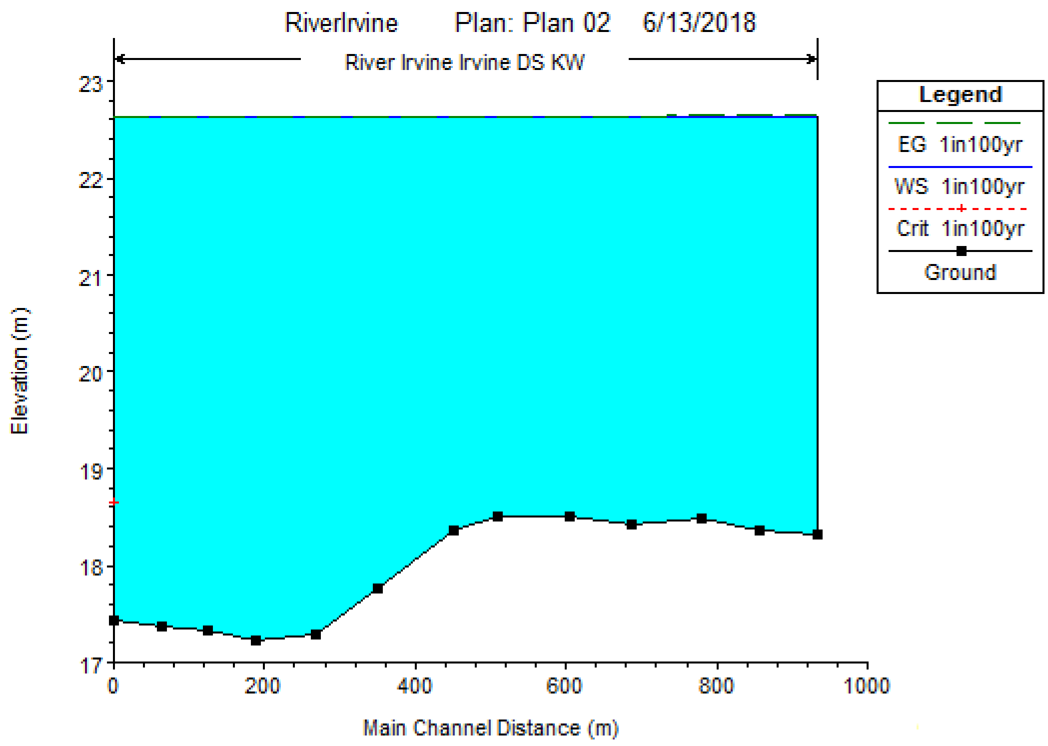

Figure 30. First profile for River Irvine DS KW

Since the definition of River Irvine DS KW was already accurate, after adding cross sections it is seen a softener dotted line which gives precision enough for no needing extra cross sections.

The final geometry with the cross sections that will be used for the flood risk analysis is the following:

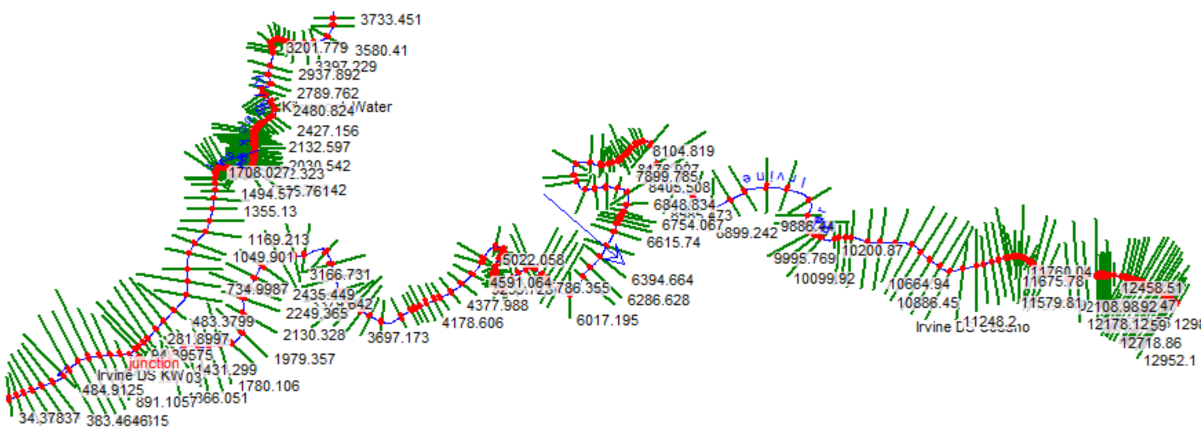

Figure 31. Final geometric data

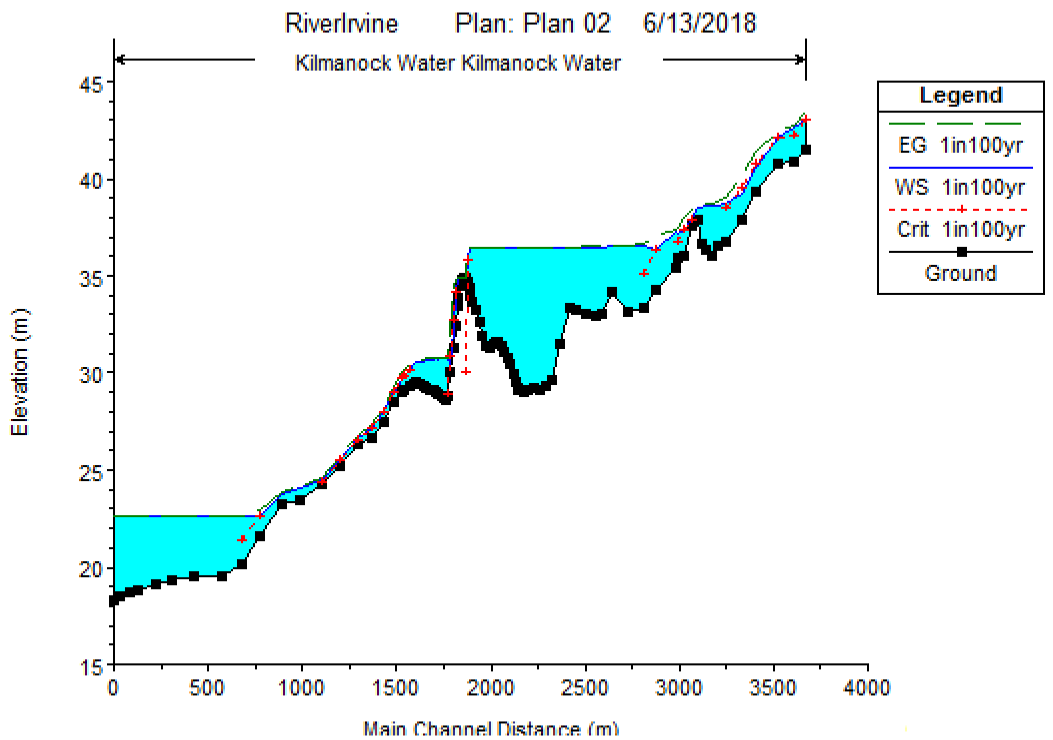

Figure 32. Final profile for Kilmarnock

Once the final geometry with its corresponding cross sections is integrated into the program, it is need to introduce Manning's roughness coefficient values for each ones of the sections previously defined.

The Manning's roughness coefficient adopted for the riverbed of the upper river sections has been calculated using Chezy's equation:

$$V = \frac{1}{n} R_h^{\frac{2}{3}} * S^{\frac{1}{2}}$$

Where:

V: Average flow velocity (m/s)

R_h: Hydraulic ratio (m)

S: Slope (m/m)

n: Manning's roughness coefficient

3.2.2. Flow data

Here the flow data and boundary conditions are introduced to the program. In this case the peak flow rate for 10, 100 and 500 years returning period obtained previously are imported.

The boundary conditions depend on the flow's regime. For supercritical flow the boundary condition is imposed upstream. For subcritical flow the boundary condition is imposed downstream. In this case the ideal is to work under mixed regime, therefore upstream and downstream boundary conditions are imposed.

These conditions can be the riverbed slope, the flow depth, the critical flow depth or a curve which relates the flow depth with the flow rate.

3.3. Results obtained using HEC-RAS

The results from executing the software HEC-RAS can be examined in the corresponding annex. With the flooding results obtained, the flooding sheets are executed defining three areas: river area, water system and flooding area.

4. CONCLUSIONS

During the execution of the present study it has been observed the lack of information and regulations provided by the Scottish and UK authorities was one of the biggest cons. There is no access to the rainfall historical records since they are all performed by private companies which are doing flood risk reports. Therefore, for obtaining this information interpolation of the few available data and Gumbel's curve were performed.

Furthermore the lack of a proper regulation defining a buffer zone method in order to elaborate the risk assessment was surprising, specially on an island located in the Atlantic with very intense rainfalls and a considerable historical record of flood events.

From the elaboration of the present flood risk report the following conclusions are presented:

- The flooding areas for the 10, 100 and 500 years returning period required by the Scottish Planning Policy have been defined in the area of interest.
- The urban area of Kilmarnock it is located on a flooding area according to SPP7: Planning and Flooding with a "medium to high risk area". For that reason, the necessary flood preventions should be provided and also the development of a long term strategy in a structure plan context should be done.

In addition, and due to the result of the conclusions, it is recommendable to use the present study in order to elaborate a Flood Management Plan in the area and invest come capital n prevention and protection against flooding episodes.

Once this study has been performed, the final conclusion is that during the process of doing the project I have learned self-taught how to use programs like HEC-HMS or Global Mapper, and at the same time to extend my knowledge of HEC-RAS and AutoCAD Civil 3D which I already started learning on the university.

5. REFERENCES

- Flood Risk Management (Scotland) Act 2009: Ayrshire Local Plan District. Local Flood Risk Management Plan. June 2016.
- Scottish Planning Policy (SPP) 7. February 04, 2010.
- Section 6 “Directrices de preservación frente a riesgos de inundaciones”. Reglamento de urbanismo, decreto 305/2006 de la Generalitat de Catalunya, 18 de Julio del 2006.