WESTMEATH
COUNTY COUNCIL

AL RIVER IMPROVEMENTS

PRELIMINARY REPORT

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AL RIVER IMPROVEMENTS SCHEME

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APPENDICES

1.0 INTRODUCTION

1.1 A1 River

The Al River rises in the townland of Crosswood approximately 3km east of Athlone and flows in a south westerly direction for a distance of approximately 5 km eventually entering the river Shannon downstream of the weir at Golden Island. The river flows through the Willow Park housing estate where it has been culverted under the estate roads. The section of the river within the housing estate is of particular interest to Westmeath County Council as there is a history of regular flooding within the estate.

This report sets out the findings of a study of the existing river channel and recommends improvements to alleviate the flooding and to provide adequate capacity for runoff from existing and future development

1.2 Terms of Reference

Westmeath County Council appointed P.H. McCarthy & Partners to prepare a report on the basis of the proposal set out in P.H. McCarthy's letter to Westmeath County Council dated 6th March 2000 The proposal considered the scope of the work as follows

- Review all existing information using currently available data e.g. maps, drawings, etc.
- Locate and assess the condition of existing surface water culverts and drains
- Identify the catchment characteristics e.g. housing, industrial, percentage imperviousness, density of housing, etc.
- Calculate the future run off from the catchment and prepare a design for a new culverts/channels.

A large area of land in the Al-river catchment is zoned for development. This development will have an impact on both the quantity and quality of the runoff. The quantity of the runoff will be increased and the time to peak will be reduced. Thus a given flow will occur with increased frequency compared to the predevelopment scenario.

This report deals mainly with the hydraulic assessment of the river upstream of the culvert under the railway at Kilmacuagh and the proposed diversion and culverting of the river from the upstream side of the culvert under the Dublin Road (N6) at Garrycastle.

1.3 Review of Existing Information

The Preliminary Report on Athlone Main Drainage prepared by P H McCarthy & Partners in 1980 included an assessment of the capacity of the river and recommended regrading the open channel and the upsizing of a number of culverts under roads.

A further Report prepared by Westmeath County Council in 1994 recommended the diversion of the upstream section of the river away from its present route through Willow Park to a new pipe/ culvert through open land to the east. In 1998 Westmeath County Council also prepared maps for the acquisition of the necessary wayleaves for the diversion of the river between the road culvert at Garrycastle (S22) and the culvert under the railway at Kilmacuagh (S24).

1.4 Existing Surface Water Culverts and Drains

The river is culverted at a number of locations along its length which function as control points in the catchment. These are as follows:

S100	Culvert under disused railway line
S101	Culvert under Athlone Relief Road
S21	Culvert under service road in the IDA Industrial Estate at Garrycastle
S22	Culvert under the Dublin Road (Garrycastle Bridge)
	Culverts under the housing estate roads at Willow Park
S24	The culvert under the railway at Kilmacuagh

2.0 CATCHMENT DESCRIPTION

2.1 Extent of Study

The river rises in a rural bog environment and flows through agricultural land before it enters an urban area comprising industrial estates and housing areas. The boundaries of the catchment are determined by the ground contours and the drainage network. It was noted during the initial walk over survey of the area that the upper extent of the catchment is determined by the constructed network of open drains in the bog being harvested.

The Al River catchment upstream of the railway culvert at Kilmacuagh is approximately 6.03km^2 in area and 5.7km in length. For the purposes of analysis the catchment was divided into sub-catchments designated SCR, SC1 and SC2.

2.2 Sub-Catchment SCR

The upper part of the catchment under consideration is currently undeveloped, containing only a few sparsely distributed dwellings, and lies outside the draft development plan area for Athlone. This area is 4.20 km^2 in area and is designated rural sub-catchment SCR. It is mostly bog-land in nature (photo 100 – 103 Appendix A), but downstream of the culvert under the disused Athlone-Mullingar railway it is agricultural land with a low gradient (photo 104 Appendix A). The river passes under the disused railway (S100 on drawing 685/1/001) through a stone arched culvert.

2.3 Sub-Catchment SC1

The part of the remaining catchment upstream of Garrycastle Bridge on the Dublin Road (S22 on drawing 685/1/001) is designated sub-catchment SC1. This sub-catchment, which is 0.94 km^2 in area, lies entirely within the draft development plan area. At present, approximately 0.34 km^2 of the sub-catchment is developed, but the remainder is zoned for industrial development. The service road in the IDA Athlone Business Park crosses the Al River (S21 on drawing 685/1/001). There is currently a temporary throttle at S21 resulting in the utilisation of storage immediately upstream of the throttle, (photo 105 – 107 Appendix A). There is capacity for online storage both immediately upstream and downstream of S21 (photo 105 – 111 Appendix A and drawing 685/1/101), with a maximum difference of 3.67m between the bed and bank level.

2.4 Sub-Catchment SC2

Downstream of the Garrycastle Bridge the remaining catchment is designated sub-catchment SC2. Sub-catchment SC2 is 0.89 km^2 in area and lies entirely within the boundary of the Athlone draft development plan, in which it is zoned for commercial, residential and educational development. At present, approximately 0.33 km^2 of the sub-catchment is developed. Immediately downstream of Garrycastle Bridge, the river traverses undeveloped land before entering the Willow Park Housing Estate (photo 115 - 120 Appendix A).

It is proposed to divert the Al River away from its current course through the Willow Park Housing Estate (see section 4.0). The proposed culvert will pick up flow at the Garrycastle Bridge and, passing east of the housing estate, will cross the Athlone-Dublin railway approximately 350m east of the existing culvert at Kilmacuagh (S24 on drawing 685/1/001). As runoff from sub-catchment SC2 would be picked up by the new culvert at different points, the sub-catchment was further divided into sub-catchments SC2a (23ha in area, 0.5ha developed at present), SC2b (18ha in area, 9.3ha developed at present), SC2c (37ha in area, 23.4ha developed at present) and SC2d (11ha in area, undeveloped at present).

3.0 CATCHMENT RUN-OFF

3.1 Run-off Calculation

The calculation of run-off from the catchment was assessed using methods set out in the Flood Studies Report (FSR) first published in 1975 subsequently updated by a number of supplementary reports, which included data relevant to Ireland. A further publication *The Flood Estimation Handbook* was published in 1999.

In the FSR rainfall-runoff method, a rainfall input is converted to a flow output using a deterministic model of catchment response. The model used is the unit hydrograph and losses model, which has free parameters. The parameters relate to the catchment response to rainfall (unit hydrograph time-to-peak), the proportion of rainfall, which directly contributes to flow in the river (percentage runoff), and the quantity of flow in the river prior to the event (baseflow). Where possible, the model parameters are derived from observed rainfall and runoff records. However, if no records exist, the model parameters may be estimated from physical and climatic descriptors of the catchment.

Once the model parameters have been derived for a catchment, the method may be used to estimate the total flow from any rainfall event. The rainfall will be in the form of a hyetograph, defined by a duration, depth and profile. The rainfall may be a statistically-derived design event to produce a flood of a specific return period (the T-year flood), or may be a probable maximum precipitation (PMP) to produce a probable maximum flood (PMF). Alternatively, the rainfall may be an observed event, the aim being to simulate a notable flood.

In the T-year design case, the duration of the design storm is related to the speed of catchment response, and the point rainfall depth is estimated for a return period which depends on the return period of the design flood. An aerial reduction factor is applied to give the catchment rainfall depth. This is subsequently transformed into a hyetograph by a standard time profile. Estimation of the PMF follows a similar procedure, with conservative assumptions regarding catchment response and the rainfall, and possibly snowmelt, inputs. For reconstruction of an event, direct estimation of catchment rainfall from observed data is possible.

In each case, the proportion of rainfall which directly contributes to flow in the river (the effective or net rainfall) is adjusted according to the runoff potential of the catchment, the rainfall total and the antecedent catchment wetness. Again, conservative assumptions about runoff potential and antecedent catchment wetness are made for estimation of the PMF, and direct estimation of antecedent condition from observed data is made for simulation of an event. The effective rainfall is combined with the catchment unit hydrograph (a process known as convolution) to form the rapid response runoff hydrograph. Finally, the flow in the river prior to the event is added, to complete the design flood.

As there is no historical flow data available for the Al River deterministic methods were used to calculate flow volumes. For comparison, estimates of flood magnitudes for a selection of time periods are calculated using a number of different methods.

The river catchment was divided into a number of subcatchments for pre-development and post development scenarios examined. In the post development phase one hundred percent urban development was assumed so that a worst case scenario could be developed.

The runoff estimation procedures are divided into two categories. The first category calculates the peak rate of flow only and the second calculates the runoff hydrograph. A flood peak discharge is used to assess channel capacity whereas a flood hydrograph is of use where the flood volume is of use in addition to a peak discharge.

Four methods were used to calculate the peak rate of flow, these were FSR 6 variable equation, FSR 3 and 4 variable equation, FSR urbanised method used for estimation of runoff for rural subcatchments, and the Rational Method used for calculations for urban subcatchments. The hydrograph method employed was the FSR Rainfall-Runoff Method for rural subcatchments and Rational Method for urban subcatchments. The results are detailed in Table 1.

A series of flow hydrographs were obtained for a range of rainfall durations up to 24 hours for the twenty year storm and flood event. The durations examined were the time of concentration (or time to peak), 0.5 hours, 1 hour, 6 hours, 13 hours and 24 hours.

3.2 Peak Discharge Methods

3.2.1 FSR 6 – Variable Equation

Methods for estimating the mean annual peak flow are described in the Institute of Hydrology Report No. 124. 'Flood estimation for Small Catchments'. The Flood Studies Report (FSR) summarises regression equations linking mean annual peak flow, Q_{BAR} on an ungauged catchment to a defined number of catchment characteristics. The best known of the several equations is the so-called 6-varaible equation.

The Catchment Characteristics Equation (CCE) is used typically in an ungauged catchment. Cunnane and Lynn in their review of the 1975 Flood Studies Report stated that in two-thirds of catchments, the percentage error would fall between +50% and –33%. Despite this the CCE is understood to give a first estimate of the catchment response for a given return period. The equation is as follows

$$Q_{BAR} = 0.0172 * AREA^{0.94} * STMFRQ^{0.27} * S1085^{0.16} * SOIL^{1.23} * RSMD^{1.03} * (1 + LAKE)^{-0.85}$$

3.2.2 FSR 3 and 4 – Variable Equations

FSR No.6 provides Q_{BAR} equations for possible use on catchments of less than 20km^2 These are as follows:

Equation FSR 3 - $Q_{BAR} = 0.00066 * AREA^{0.92} * SAAR^{1.22} * SOIL^{2.0}$

Equation FSR4 - $Q_{BAR} = 0.0288 * AREA^{0.90} * RSMD^{1.23} * SOIL^{1.77} * STMFRQ^{0.23}$

- Q_{BAR} is the mean annual flood for the catchment area
- AREA is the area of the catchment in km^2
- STMFRQ is the stream frequency of the catchment calculated by counting the number of channel junctions and dividing by AREA
- S1085 is the channel slope (m/km) obtained from the average slope between points located 10 and 85% of the main channel upstream from the catchment outlet
- SOIL is the soil index derived from Figure I.4.18 (FSR Vol.5 - Maps, 1975) in which all soils are allocated to one of five classes according to their physical properties and topographic features

- RSMD is the 5 year, 1 day rainfall excess (mm) obtained using SAAR
- SAAR is the average annual rainfall (mm)
- LAKE is the lake index defined as the fraction of the catchment draining through lakes or reservoirs

There is no term in these equations to represent the degree of urbanisation.

3.2.3 FSR Urbanised Method

FSSR No.5 provides a means of extending Q_{BAR} estimation to partly urbanised greenfield catchments which the 6 variable equation does not deal with. It presents revised procedures to account more satisfactorily for urbanisation particularly relevant to catchments in the 5 to 100km^2 size range and where urban development is fairly uniformly distributed over the catchment. Specifically, the effect on mean annual peak flow may be estimated from

$$Q_{BARurban} / Q_{BARrural} = (1 + URBAN)^{1.5} (1 + 0.3URBAN (70/PR_{rural} - 1))$$

- $Q_{BARurban}$ and $Q_{BARrural}$ refer to the urban and rural conditions of the greenfield site respectively.
- URBAN is the fraction of the catchment that already is or is to be urbanised
- PR_{rural} is the percentage runoff for the catchment in its rural condition

Once the average annual discharge from the greenfield catchment has been calculated by one of the above methods, the T-year discharge event from the greenfield catchment can be calculated.

3.2.4 Rational Method

The Rational Method of flood estimation is used particularly in the specialised hydrology of urban drainage systems.

The basis of this method is a simplistic relationship between rainfall and runoff given below:

$$Q_p \ (ft^3/s) = C \ i \ (in./h) A(acres)$$
$$Q_p \ (m^3/s) = 0.278C \ i \ (mm/h) A(km^2)$$

- Q_p is the peak discharge due to particular rainstorm, and assumed to occur after time t_c when the whole catchment area is contributing
- t_c is time of concentration, i.e. the time taken for rain falling on the catchment farthest from the gauging station to arrive there and is usually given as sum of the time to entry into the drainage system (t_e) and the time of flow through the drainage system (t_f)
- C is the dimensionless runoff coefficient, whose value depends on the catchment characteristics (usually between 0.1 and 0.9)
- i is the rainfall intensity for a storm duration t and is assumed to be uniform over the catchment. The rainfall intensity can be obtained from an appropriate intensity-duration-frequency table for the site location.
- A is the drainage area contributing flow.

3.3 Hydrograph Methods

3.3.1 FSR Rainfall - Runoff Method

FSR Unit Hydrograph and Losses Model

The 3-parameter unit hydrograph and losses model forms the core of the FSR rainfall-runoff method.

If no flow records exist, the synthetic unit hydrograph can be estimated from catchment characteristics alone.

3.3.2 Derivation of the Unit hydrograph

The t-hour unit hydrograph is assumed to be triangular in shape, with a peak discharge Q (m^3/s/10mm of net rainfall) occurring at the TP_t(h).

Given the catchment characteristics, the time of rise, TP_o(h), of the instantaneous unit hydrograph is computed from:

$$TP_o = 283.0(S1085)^{-0.23}(1 + URBAN)^{-2.2}(SAAR)^{-0.54}(MSL)^{0.22}$$

where MSL is the main channel length.

There is a term to represent urban runoff but the spatial location of the urbanisation is not represented. Thus the fact that an urban development located on the downstream end of the river catchment will be less likely to increase river flow magnitude than one further upstream is not accounted for.

The data interval (which is also the period of the unit hydrograph) is estimated from t = TP0/5, rounding to the nearest multiple of one hour. The time of rise of the t-hour unit hydrograph. TPt(h) and Qp then follow from:

$TP_t = TP_o + t/2$

$Q_p = (220/TP_o)/(AREA/100)$

3.3.4 Derivation of the Net Rainfall Profile

The design storm duration, D (h), is obtained from:

$D = TP_t\ (1 + SAAR/1000)$

Rounded to the nearest odd integer multiple of t. using this value of D and the selected return period T (years), figures and tables in the FSR are used to give the design rainfall depth, P (mm). The percentage runoff, PR_{TOTAL}, is calculated from several variables found in the FSR.

The total rainfall depth is distributed over the duration D, using the 50% summer rainfall profile from the FSR and the ordinates multiplied by $PR_{TOTAL}/100$ to give the net rainfall profile.

3.3.5 Computation of the Design Flood Hydrograph

The net rainfall profile is applied to the unit hydrograph to give the surface runoff hydrograph and the design flood hydrograph is obtained by adding a constant baseflow to the ordinates of the surface runoff hydrograph. The baseflow, ANSF ($m^3/s/km^2$) is computed from:

$ANSF = [33(CWI – 125) + 3(SAAR) + 5.5]*10^{-5}$

Which is multiplied by AREA to give the baseflow in m^3/s.

CWI represents catchment wetness index and is a function of the average annual rainfall (SAAR).

3.4 The Wallingford Procedure Rainfall - Runoff Model

The Wallingford Procedure rainfall-runoff model is used in HydroWorks for calculation of the runoff from urban areas to the drainage system.

The hydrological elements of the urban catchment model represent the catchment characteristics to enable the calculation of the volume of runoff for a given event and the inflow profile to each pipe.

HydroWorks separates the two parts to runoff: the proportion of rainfall contributing to the system and the routing of the water into the network. Initial rainfall losses from depression storage is taken into account before rainfall runoff is assumed to take place.

The model was calibrated against a large number of events recorded in UK and is a regression equation. The most important parameter is the paved percentage of the catchment followed by a factor referred to as SOIL (soil characteristic) and UCWI (Urban Catchment Wetness Index).

$$PR = 0.829PIMP + 25SOIL + 0.078UCWI - 20.7$$

Where:

- PR is percentage runoff
- PIMP is percentage impermeability (0-100)
- SOIL is an index of the water holding capacity of the soil (0.15-0.5)
- UCWI is Urban Catchment Wetness Index (0-300) which value is linked to the SAAR (Standard Annual Average Rainfall) value for that location.

The model predicts the total runoff from all surfaces in the subcatchment, both pervious and impervious. Runoff for the entire catchment is distributed between the different surfaces using weighting coefficients. All surfaces can therefore contribute some runoff even at low runoff rates, provided that initial losses have been satisfied.

The weighting is carried out as follows:

$PRi = f_iA_i/(f_1A_1+f_2A_2+f_3A_3)*PR$

- f_1, f_2, f_3 are weighting coefficients for paved, roofed and pervious surfaces and their default values are 1.0, 1.0 and 0.1 respectively

3.5 Results

3.5.1 Design Storm Depth

The design storm depth P is the T-year D-hour catchment rainfall. The storm depth P is determined from rainfall depth – duration – frequency relationships (Table1). Determination of the appropriate rainfall return period depends on the degree of urbanisation of the catchment and the required return period of the flood. On rural or only moderately urbanised catchments, the design rainfall return period T_R is determined from the design flood return period T_F using the graph in Figure 2. On urban catchments the design rainfall period T_R is set equal to the design flood return period T_F. The reasoning behind this is that for rural catchments, because of other factors (e.g. antecedent condition), not all extreme rainfalls generate equal extreme floods; however, urbanised catchments are generally less variable in their response, making a simpler choice of design conditions possible.

The 20 year return period is considered appropriate for the design of the culvert. For the rural subcatchment the FSR methodology was applied. Taken from the graph in Figure 2, the corresponding rainfall event for the flood of 20 year return period, is of 34 year return period. This was used to produce series of flood hydrographs with different storm durations. The critical rainfall that produces the flood peak flow for the rural subcatchment is of 13 hours duration for predevelopment as well as for post development stage and flood peaks are 0.99 m^3/s and 0.94 m^3/s respectively (Appendices A).

The urbanisation is not uniformly spread about the catchment, it is concentrated in downstream areas. Therefore, its influence on the peak of the runoff at the outlet of the catchment is more significant and for such heavily urbanised i.e. industrial subcatchments it is recommended to treat flood estimation by sewer design methods. The rational method explained in section 3.1 is used. The calculated critical storm is of 30 minutes duration (Appendices A).

3.5.2 Runoff

There is a difference in the response (runoff quantity and timing) of the urban type impermeable areas and the rural agricultural areas to rainfall events. The response from rural areas is relatively slow and typically produces peak runoff after prolonged periods of rain. The response from the urban or industrial type impermeable areas is very fast and the peak flows essentially depend on the distance of the farthest end of the subcatchment to the outlet and the rainfall intensity. In this particular case urbanisation is set in downstream areas of Al River catchment which is of interest. This will cause the urban response to pass before the slow rural response from upstream arrives, thus determining the peak discharge from urban subcatchment as critical one.

The results of the runoff calculations using the various methods are summarised in Table 2 (Appendix A).

The HydroWorks software package for hydraulic modelling of sewerage systems was also used. The model was built as Skeletal Planning Model to provide overall assessment of a whole catchment to consider what effects a major new development might have on the river and downstream areas. The model consists of nodes, which correspond to junctions or manholes between two or more conduits in the physical network and links that correspond to one or more conduits in the physical network. The river was modelled as an open channel. Urban subcatchments SC1 and SC2 were further subdivided to subcatchments of approximately 5 ha in area, each draining to one node. Thus, the future storm sewer network was represented. The model reproduces the temporal and spatial distribution of rainfall over the catchment being drained by the network as well as the simulated runoff over subcatchment surfaces into the corresponding manholes. The rainfall – runoff relation is sensitive to a number of parameters including paved area, soil type, catchment wetness, size of contributing area, and surface gradient. Besides accommodating rainfall – runoff to a node, the model can also discharge flows back onto the surface, such as when the network is under excessive surcharge. The resulting surface flooding can be represented in the model as well as its effects on the continuing flow in the network.

The simulations were run with generated storms of different durations and the critical rainfall was of 30 minutes duration. Two types of model were used: one with all conduits including river channel sized sufficiently to convey all runoff flows to assess total runoff from the future paved areas, and second model without any changes to the river channel to represent the future condition if the river channel and banks characteristics remain the same. The estimated runoff from the catchment without making allowance for flooding and storage is 10.95 m^3/s. This flow was derived using the two models described above. The existing river channel and culverts, act as a throttle at present and if these were to remain unaltered the flow downstream of Garrycastle bridge would be restricted to $7.3m^3/sec$. However this would result in excessive flooding along the river (Appendix A).

The deficiencies in the existing system are as follows

- The existing open channel and culverts through the Willow Park/Meadowbrook Housing Estate do not have sufficient capacity to carry the existing and future storm flows.

- The capacity of the existing culvert under the railway line is $3.40m^3/s$ however the full capacity cannot be utilised as the soffit of the culvert is at a level of 40.28m.OD which above the floor level of the houses closest to the culvert.

- The existing river channel and culverts downstream of the railway do not have sufficient capacity to carry future flows

4.0 PROPOSALS

4.1 Options

In order to resolve the difficulties caused by the regular flooding in the Willow Park/Meadowbrook Housing Estate and to provide for the drainage of future development areas we considered the following options.

Option 1

- Replace the existing culverts and upgrade the channels through the Willow Park/Meadowbrook Housing Estate
- Enlarge the existing culvert under the railway
- Replace the existing culverts and upgrade the channel downstream of the railway

Option 2

- Divert flows away from the Willow Park/Meadowbrook Housing Estate through a new culvert or open channel
- Construct a new culvert under the railway
- Replace the existing culverts and upgrade the channel downstream of the railway

Option 3

- Divert flows away from the Willow Park Estate a new culvert
- Construct a new culvert under the railway
- Construct a new channel through open bogland to the Shannon to divert flows away from existing channel downstream of the railway.

Option 3 is considered the best option for the following reasons

The new culvert required to divert flows away from Meadowbrook/Willow Park can be constructed through open undeveloped land east of the estate. This will be much less difficult and cheaper than constructing new culverts within the Estate.

The existing channel downstream of the railway flows through the urban area, any enlargement of the channel and the necessary construction of flood banks will restrict further development in the urban area. The diversion of flows through a new channel through land not earmarked for development will facilitate future long-term development in the urban area.

4.2 Proposed Scheme

The proposed scheme will comprise the following:

Culvert between N6 and Railway Line

All existing river flows and future run-off from the catchment north of the National Primary Route N6 and the development area between the road and the railway will be carried in a new culvert running between the northern side of the N6 and the southern side of the railway line (see drawing 685/1/106). The culvert will consist of twin 1500mm dia. pipes or equivalent. The precise route of the culvert between the N6 and the railway will be determined by the layout of future development in this area. However, it is envisaged that it should be relatively easy to accommodate the culvert within the line of the future road or public open space.

Culvert under the Railway

The culvert under the railway will be twin 1500mm dia pipes. The construction of the culvert will pose particular difficulties and will require tunnelling or open cut excavation during a short-term closure of the railway. We have met with representatives on Iarnrod Eireann on site to discuss the matter and they have indicated their approval in principle to the crossing of the railway.

The construction of the culvert under the railway is further complicated by the proposed construction of a gas pipeline between Dublin and Galway. The proposed route of the pipeline passes along the south side of the railway close to the proposed culvert crossing of the railway and will clash with the line of the proposed culvert

The gas pipe, which is steel 750mm dia. constructed at a depth of approximately 2.0m. We have discussed the details of the crossing with M.C. O'Sullivan & Co. Consulting Engineers for Bord Gais. The gas pipe will have to be lowered to pass beneath the proposed culvert and the crossing will require piling of the culvert and the gas pipe. Finalisation of the design of the crossing is being considered as a matter of urgency as the programme for the gas pipeline construction envisages this section being constructed in the next few weeks.

Open Channel through Bogland

The new culvert under the railway will discharge to a new channel constructed generally along the northern edge of an area of bogland in the townlands of Derries and Bunahilly and through lands to the west of the bog to one of the open drains which form a network of open drains, which connect to the Shannon at Carrickobreen. The existing open drain will be modified to cater for the increased future flow. Typical cross sections of the channel through the bogland are shown on drawing No. 685/01/114.

The channel will need to be lined to provide stability to the banks in the soft peat and to prevent the ingress of peat, which would result in silting and require regular maintenance. There is a number of proprietary lining systems available, which would be suitable. Our design provides for the use of a geosynthetic clay liner *Bentofix*, which utilises sodium bentonite clay. Technical data relating to *Bentofix* is included in Appendix D.

All runoff from the surrounding bogland will be excluded from the channel and this will require the construction of secondary open drainage channels parallel to the channel.

Land Acquisition

The construction and functioning of the channel will require the purchase of a strip of land approximately 40m wide which should be fenced to provide protection of the channel profile and allow for future maintenance. In times of Shannon flood the channel will fill to ground level in common with all other channels within the flood plain of the river.

Cost Estimate

We estimate the cost of constructing the culvert and the open channel to be €4,180,171. The make up of this amount is set out Appendix E.

5.0 STORMWATER MANAGEMENT POLICY

5.1 Stormwater Management

The maximum permitted surface water outflow rate from a future new development may be specified as:

a fixed flow rate
a fixed flow rate per hectare of catchment drained
a variable flow rate depending on storm duration

the estimated green field runoff from the catchment for a storm of specified return period.

In broad terms, control of runoff can be achieved in two ways:

reduction of flows entering surface water sewers or watercourses

attenuation of flows either before entry to the surface water sewer or within the downstream drainage system so that the available capacity of the system is utilised and the need from further capacity deferred or avoided.

A reduction in volumetric input to downstream surface water sewers or watercourses can result from one or more of the following courses of action:

infiltration of urban runoff to subsurface strata

controlled inlet to surface water sewers or watercourses, resulting in a reduction in the peak flow and, in some instances, reducing the volume of runoff.

In broad terms, control of runoff by attenuation methods requires a hydraulic control to restrict the magnitude of flows passing downstream, together with an upstream storage capacity to contain the volume of runoff held back by the hydraulic control.

Attenuation of flows within the drainage system can be divided into three types:

upstream of the surface water sewer system

within the surface water sewer system

within the watercourse.

The attenuation of flows within the surface water sewerage system for development provides an economical solution to reduce peak discharge rates so that the existing downstream sewerage system or receiving watercourse can cope. The alternative, which is to uprate the capacity of the downstream surface water sewer or watercourse to accept increased flow, can be both expensive and disruptive. Controlling flows within the surface water system requires excess flow to be stored until the downstream surface water sewerage or channel system has spare capacity.

Wherever possible design of these controls should seek to ensure that the first flush after a dry period is retained and released slowly preferably by infiltration. This will prevent rapid runoff from minor summer storms and reduce pollution.

Maintenance of the systems should be planned for. Source control measures should be designed to minimise the pollution of groundwater.

5.2 Hydrological Impacts

With increased paved area and consequent reduction in impervious surfaces the volumes and rates of runoff increase significantly. The higher frequency of peak flows causes the urban stream to cut a deeper channel, effecting the stability of banks, marginal habitats and the amount of sediments in suspension.

During prolonged dry periods there will be a reduction in residual seepage from developed urban areas to maintain low flows. This will results in a further degradation of the river habitat.

5.4.3 Water Quality

The more pollutants used in a catchment, the greater the risk of diffuse pollution. This would indicate that a watercourse traversing an industrial area is particularly vulnerable. Urban development can be responsible for increases in BOD and COD levels, bacteria, and can produce additional loadings of organic plus ammonial nitrogen, inorganic particles (grit) and heavy metals (such as copper, zinc and cadmium).

Best management practices in the management of stormwater runoff should be instigated in the areas zoned for redevelopment to reduce the risk of diffuse pollution reaching the watercourse and subsequently the river Shannon. Samples should be taken for water quality analysis to establish baseline environmental data before development commences. The issues arising from this are beyond the scope of the present study.

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

The response from rural subcatchment areas is relatively slow and typically produces peak runoff after prolonged periods of rain. The response from the urban and new development areas will be much faster. The urban flows will therefore pass before the slower rural flows from upstream arrive, thus determining the peak discharge from urban subcatchment as critical one.

The deficiencies in the existing system are as follows:

- The existing open channel and culverts through the Willow Park/Meadowbrook Housing Estate do not have sufficient capacity to carry the existing and future storm flows.
- The capacity of the existing culvert under the railway embankment is $3.40m^3/s$ however the full capacity cannot be utilised without causing a serious risk of flooding. As the level of the land and the houses immediately upstream of the culvert is close to the level of the soffit of the culvert.
- The existing river channel and culverts downstream of the railway do not have sufficient capacity to carry future flows

6.2 Recommendations

Having examined the Al River catchment as it is at present and considered the impact future development would have on the existing flows in the river we recommend the following measures to alleviate flooding in the Willow Park/Meadowbrook Housing Estate and to provide for the adequate drainage of future development within the catchment.

- The construction of a new culvert to divert flows away from the Willow Park Estate.
- The construction of a new culvert under the railway
- The construct a new channel through open bogland to the Shannon at Carrickobreen

Signed: ________________________

P.H. McCarthy & Partners

APPENDIX A – CATCHMENT RUN-OFF CALCULATIONS

APPENDIX B – PHOTOGRAPHS

APPENDIX C – LIST OF REPORT DRAWINGS

APPENDIX C

LIST OF REPORT DRAWINGS

Drawing No.	**Title**
685/01/001	Catchment Map
685/01/101	Plan & Longitudinal Section of Existing Channel
685/01/102	Plan of Existing Channel showing Photograph Locations
685/01/106	Plan & Longitudinal Section of Culvert at Kilmacuagh
685/01/108	Plan & Longitudinal Section of New Culvert and Open Channel from Garrycastle Bridge to Shannon
685/01/114	Cross Sections of New Channel through Bogland
685/01/115	Detail of Channel Liner

APPENDIX D BENTOFIX TECHNICAL DETAILS

APPENDIX E – COST SUMMARY