APPLICATION FORM AF-50:
CONSENT UNDER SECTION 50, ARTERIAL DRAINAGE ACT, 1945

1.APPLICATION DETAILS				
Name of Applicant:				
Company / authority:				
Address:				
Date of Application:				
Client (if appropriate):				
BRIDGE DETAILS				
Bridge Name:	**Structure ST01 Latoon Creek Bridge Ch. 12030**			
Purpose (ring appropriate box)	**Public Road**	Private Road	Footbridge	Other
Road Number (or Name): **N18**				
River: **Ardsollus River**		Catchment: **160 km²**		
County: **Clare**		Grid Reference: **137918,171920**		
Location:	**Dromoland**			
Type of Works (ring appropriate box	**New Bridge**	Replacement Bridge	Alterations	

OPW standards for Section 50 consent have been revised since the submission of this application. This application is for illustrative purposes only. Some amendments and annotations have been made. No drawings have been provided with the examples. Please refer to the current Section 50 brochure for current standards.

This document is provided for guidance only. The standard required for applications for this type of bridge may have changed materially since this application was approved.
This has been edited by the Office of Public Works.

1.Introduction

Latoon Creek refers to the lower reach of the Ardsollus River at the point of its confluence with the River Fergus, about 4 km downstream of Clarecastle. The proposed N18 Bypass will cross Latoon Creek upstream of the existing N18 bridge crossing and immediately upstream of Crow's Bridge. At the proposed crossing location, the Ardsollus River has a catchment area of 160 km^2. Tidal effects extend for approximately 2 km upstream of the proposed crossing location.

During 1999, the Department of Engineering Hydrology at the National University of Ireland, Galway completed a first order assessment of the hydraulic impact of the proposed Latoon Creek Crossing, the results of which were summarised in a report entitled: "Hydraulic Impact of the Latoon Creek Crossing". (A copy of the report is attached as Appendix A.)

As part of the present study, the findings of the above report were refined through the application of additional hydrological and hydraulic analysis methods. However it is important to note that, similar to the previous report, the results of the analyses described below should only be considered as first order estimates as the analyses did not involve detailed hydraulic modelling. Furthermore, all data relating to historic flood levels and cross section dimensions as per the National University of Ireland report were accepted as correct.

2.Design Flood

The Office of Public Works (OPW) requires the analysis of the 200-year situation for tidal sites. This would include fluvial floods, tidal floods and appropriate combinations using joint probabilities. Following discussions with the OPW, for the purposes of this present application the combination chosen of the 200-year fluvial flood and the extreme tide of 1999 will suffice. A sensitivity analysis was also undertaken.

For this study, the 200-year fluvial flood was used. To accommodate normal tidal flow contribution, an ebb flow rate of 40 m^3/s has been combined with the 200-year fluvial flood estimate in order to derive the design flood. The tidal component was calculated as described in the report "Hydraulic Impact of the Latoon Creek Crossing" (NUIG, 1999).

No flow records exist for the Ardsollus River. To assess the 200 year fluvial flood at the study site, three different methodologies were applied: the empirical, method as described in the report "Hydraulic Impact of the Latoon Creek Crossing"(NUIG, 1999), (Note the higher soil value used in calculations), the FSR method for the estimation of the flood peaks from catchment characteristics and the standard FSR rainfall-run off method.

2.1 NUIG Empirical Method

The empirical method described in the report "Hydraulic Impact of the Latoon Creek Crossing"(NUIG, 1999) uses the following equation to calculate the mean annual maximum fluvial flood ($\overline{Q}$):

$$\overline{Q} = 0.00066(AREA)^{0.92}(SAAR)^{1.22}(SOIL)^{2} \qquad (1)$$

Substitution of the following parameter values into equation (1) leads to an annual maximum flood of 57.8 m^3/s. (Note: revised SOIL value compared to Appendix A).

AREA	:160 km^2	(catchment area – 1:50 000 OS Map)
SAAR	:1100mm	(annual average rainfall FSR Vol V Fig II 3.1(I))
SOIL	:0.4	(soil infiltration potential FSR Vol V Fig I 4.18(I))

In order to calculate the 200-year flood, $\overline{Q}$ was multiplied by a growth factor of 2.14 (FSR Vol I, Table 2.39), which provided an empirical 200 year fluvial flood estimate of **124 m^3/s.**

2.2 FRS Estimation of Flood Peaks From Catchment Characteristics

The following equation for the estimation of floods peaks from catchment characteristics (FSR Vol I, Eq. 4.14) was used to calculated the annual maximum fluvial flood ($\overline{Q}$) at the site:

$$\overline{Q} = 0.0172(AREA)^{0.94}(STMFRQ)^{O.27}(SOIL)^{1.03}(RSMD)^{1.03}(1+ LAKE)^{-0.85}(S_{1085})^{0.16} \ (2)$$

Substitution of the parameter values listed below into equation (2) leads to a mean annual maximum flood of 30 m^3/s, which after multiplication with the growth factor of 2.14 (see Section 2.1), provides an empirical 200 year fluvial flood estimate of **64m^3/s**. (A factor for standard error was not applied)

STMFRQ	: 0.5 per km^2	(stream frequency – FSR Vol I Fig 4.7)This should be verified for the individual catchment. For Ireland junctions are counted on the 1:63360 (1 inch) maps and converted to stream frequency using FSR Vol I Fig 4.6.
M5 (2 day)	: 55 mm	(rainfall – FSR Vol V Fig I 3.2 (I))
ARF	: 0.96	(area reduction factor – FSR Vol II Fig 5.1)
M5 (24 h)	: 45 mm	(rainfall = 0.85*M5 (2 day)*ARF – FSR Vol II Table 3.7)
M5 (1 day)	: 41 mm	(rainfall = M5 (24 h)/1.11 (FSR Vol II Table 3.1)
SMD	: 4.7mm	(soil moisture deficit – FSR Vol Fig I 4.19)
RSMD	: 36 mm	(M5 (1day) rainfall less effective SMD)
LAKE	: 0.01	(catchment fraction drainage through lake – 1:50 000 OS map)
S_{1085}	: 7.7 m/km	(10 – 85% stream slop – 1:50 000 OS map)

Note: Empirical equations provide an estimate of the design flood, which may be higher or lower than the actual flood for the location in question. A factor for standard error is required to ensure that there is a low probability of a low estimate, and to reduce the magnitude of the error for the small number of locations for which the design flood would still be underestimated. The need for such a factor is clear from this example.

2.3 FSR Rainfall-Runoff Method

The standard FSR unit hydrograph method was used to derive a rainfall-runoff estimate of the 200-year fluvial flood at the study site. In addition to the parameters above, the following main parameter values were estimated as part of the analysis:

Unit hydrograph characteristics

URBAN	: 0.0	(fraction of catchment urban development – 1:50 000 OS map)
MSL	: 34.5km	(main stream length - 1:50 000 OS map)
$T_p(0)$	: 7.42 h	(instantaneous time to peak – FSSR 16)
T	: 1 h	(data interval)
$T_p(1)$	:7.92 h	(unit hydrograph time to peak – FSSR 16)

Design rainfall

D	: 17 h	(design storm duration – FSR Vol I eq. 6.64)
$T_{rainfall}$	:250 a	(required rainfall return period for flood return period of 200 years – FSR Vol I Fig 6.61)
M5(17 h)	:41 mm	(= 0.75* M5 (2 day) – FSR Vol II Table3.7)
ARF	:0.93	(areal reduction factor – FSR Vol II Fig 5.1)
M250 (17h)	:95 mm	(design rainfall = 2.5*M5(17 h)*ARF – FSR Vol II Table 2.7)

Design runoff

SPR	:39%	(standard percentage runnoff – FSSR 16)
CWI	:125	(catchment wetness index – FSR Vol I Fig 6.62)
CN	:21	(curve number – FSR Vol I Fig 6.64)
PR	:46%	(percentage runoff – FSSR 16)

Substitution of the above parameter values into the FSR rainfall-runoff model leads to a 200 year fluvial flood estimate of **147m^3/s**.

2.4 Design Flood Estimate

The above analyses show that the NUIG empirical method and the FSR rainfall runoff method give 200 year flood estimates of the same order of magnitude, i.e.124 m^3/s and 147 m^3/s respectively, while the FSR catchment parameter estimate is much lower at 64 m^3/s. (The application of a design factor for standard error would decrease the divergence).

For the hydraulic analysis, the 200 year fluvial design flood peak at the study site was estimated as the average of the NUIG empirical and FSR rainfall-runoff flood peak estimates, i.e. Q_{200} = 136 m^3/s.
The combined 200 year fluvial and the normal ebb flow rate at the study site therefore equals **176m^3/s** and was used as design flood in the hydraulic analysis.

3. Hydraulic Analysis

The existing Latoon Creek channel upstream of the proposed N18 Bypass crossing location has a typical width of 90 m between levees. The proposed N18 Bypass crossing over Latoon Creek will be a single span structure with a span of 55 m between abutments. The bridge will not cause encroachment into the main river channel, but the existing flood levees will have to be realigned in order to tie in with the bridge abutments. The bridge deck will be arched with a proposed minimum soffit level of 4.9 m OD (Malin) at the abutments.

During the 1999 December floods, which was a combination of extreme fluvial flow and high spring tides, the observed water level at the location of the proposed Latoon Creek crossing, based on trash marks, was 3.75 OD. At this water level, the existing channel upstream of the study site has a flow area of approximately 260 m^2, while the flow area after the construction of the bridge will be reduced to 198 m^2 (NUIG, 1999).

In order to calculate the possible hydraulic effect of the proposed N18 Bypass bridge crossing over Latoon Creek, two methods were applied viz. the Energy Loss method and the United States Bureau of Public Roads (1970) method. Whereas the first method only addresses energy losses in the reach upstream of the bridge due to flow contraction, the second method also accommodates energy losses through the bridge and immediately downstream of the bridge.

3.1 Energy Loss Method

Essentially, the energy loss method ascribes the afflux upstream of a bridge to transitional energy losses associated with the constriction of the river channel. The energy loss resulting from the flow constriction is calculated by multiplying the change in velocity head between the existing upstream channel section and the constricted channel section with a contraction coefficient (see equation 3).

$$h_l = K_c \left| \frac{v_2^2}{2g} - \frac{v_1^2}{2g} \right| \qquad (3)$$

with h_l : transitional energy loss (m)
K_c : contraction coefficient (assumed 0.5)
v_1 : average flow velocity (upstream section) (m/s)
v_2 : average flow velocity (bridge section) (m/s)

Using the estimated flow areas of 260 m^2 and 198 m^2, at the upstream and bridge sections respectively, as an indication of the anticipated flow contraction at the bridge site, values of 0.66 m/s and 0.89 m/s are calculated for V1 and V2 respectively. Subsequently, equation 3 yields an energy loss of **0.008m** at the design flood of 176 m^3/s.

3.2 United States Bureau of Public Roads (USBPR 1970) Method.

If it is assumed that the flow through the proposed bridge will be subcritical, the anticipated increase in water level may be calculated from the following equation (hydraulics of Bridge Waterways, US Dept of transport Federal Highway Administration, 1973)

$$h_1^* = K^* \alpha_2 \left(\frac{v_2^2}{2g}\right) + \alpha_1 \left[\left(\frac{A_2}{A_4}\right)^2 - \left(\frac{A_2}{A_1}\right)^2\right] \frac{v_2^2}{2g} \qquad (4)$$

with	h_1*	: backwater effect/increase in water level (m)
	$K*$	: total backwater coefficient
	α_1	: kinetic energy coefficient in the upstream section
	α_2	: kinetic energy coefficient in the constriction
	v_2	:average flow velocity through the constriction based on the flow area below the unconstricted normal depth (m^2)
	A_2	:flow area at construction below the unconstricted normal depth (m/s)
	A_4	: flow area at section downstream of bridge (m^2)
	A_1	: flow area at section upstream of bridge (including backwater effect)

Equation 4 is often simplified by assuming that the difference in flow area between section 1 and section 4 is negligible, which leads to the following equation:

$$h_1* = K* \alpha_2 \left(\frac{V_2^2}{2g} \right) \qquad (5)$$

*K** allows for a base coefficient to which is added incremental coefficients for the effects of piers, eccentricity and skewness. For this analysis it was assumed that the proposed bridge will be a single span bridge, that eccentricity equals zero and that the bridge will be constructed perpendicular to the flow direction. Both K* and α_2 depend on the bridge opening ration (M), which is defined as the ratio of the flow which can pass unimpeded through the bridge constriction to the total flow of the unobstructed river. Because of the lack of data, the value of M was estimated as the ratio of the widths of the bridge section to the upstream cross section, which provides an M value of 0.61. Empirical graphs (Hydraulics of Bridges Waterways, US Dept of Transportation Federal Highway Administration, 1973) were used to estimate the corresponding value of K* as 0.87, while a value of 1.2 was assumed for α_2. For an estimated flow areas of 198 m^2 at the bridge section, the value of v_2 equals 0.89 m/s and substitution of the above values into equation 5 leads to an estimated increase in water level of **0.042m** upstream of the proposed bridge site for a design flood of 176 m^3/s.

4. Sensitivity analysis

As the above analysis were not based on detailed hydraulic modelling or a topographic survey, a sensitivity analysis was undertaken which involved increasing the design flood and water level at the bridge. The following scenarios were investigated:

- Increased the design flood to 195 m^3/s.
  This is based on the highest estimate of the 200 year flood (147m^3/s) plus 48 m^3/s (the ebb flow rate of 40 m^3/s increased by 20 %).
- Increased the design flood rate to 288 m^3/s.
  This is based on twice the NUIG method estimated (124 m^3/s) plus the ebb flow rate of 40 m^3/s.
- Increased the flood level at the bridge to 4.70 m OD
  To review the head loss due to the structure at an elevated tailwater.

The results of the main analysis as well as the sensitivity analysis are summarised in the Table below and show the anticipated head loss at the bridge. Note that due to the lack of cross sectional data at the site, a linear relationship was assumed between elevation and flow area on both the bridge and upstream cross section.

Design Flood	**Flood water Level**	**3.75 m OD**	**4.70 m OD**
	Method	**Blackwater effect (h_1)**	
176 m^3/s.	Energy loss method	0.008 m	0.006 m
	USBPR method	0.042 m	0.026 m
195 m^3/s.	Energy loss method	0.010 m	0.007 m
	USBPR method	0.052 m	0.032 m
288 m^3/s.	Energy loss method	0.023 m	0.016 m
	USBPR method	0.113 m	0.071 m

Table 1 : Hydraulic analysis results

Based on the above results it is estimated that the construction of the N18 Bypass Latoon Creek crossing will result in a maximum increase in the water level of 0.042 m at a design flood of 176m^3/s. If the observed December 1999 flood level of 3.75m OD is assumed to represent the existing water level at the bridge site during an extreme event, this implies that the upstream water level after construction of the N18 bypass bridge may increase to a minimum soffit level of 3.792m OD during 200 year fluvial event. The freeboard to the minimum soffit level of the bridge deck (4.9m OD) therefore equals 1.108m, which exceeds the OPW freeboard requirement of 0.3m by 0.808m.

The sensitivity analysis showed that, based on the USBPR method a design flood of 288 m^3/s will increase the upstream water level by a maximum of 0.113 m at a flood water level of 3.75m OD. The corresponding water level upstream of the bridge crossing would therefore equal 3.863m OD, which is 1.037 m below the proposed minimum soffit level of the bridge deck. Similarly, at a flood water level of 4.70m OD, a maximum increase in water level of 0.071 m at a corresponding water level of 4.771m OD, is estimated at a design flood of 288m^3/s. This level is still below the proposed soffit level, although it does not exceed the 0.3m OPW freeboard specification. The sensitivity analysis also show that the relative backwater impact is less at higher flood water levels, which may be ascribed to lower flow velocities associated with higher flood levels.

5. Conclusion

Taking into account that both the observed December 1999 flood level of 3.75 m OD as well as the 200 year design flood of 176 m^3/s represent extreme events, it is concluded from the results of this analysis that the proposed N18 bypass bridge crossing has more than sufficient freeboard. This is further substantiated by the sensitivity analysis, which showed that the anticipated increased in water level is still below the proposed soffit level, even at flows as high as 288 $m^3/$ and at flood water levels of 4.7 m OD

APPENDIX A

Hydraulic Impact of the Latoon Creek Crossing

Department of Engineering Hydrology at the National University of Ireland

Hydraulic Impact of the Latoon Creek Crossing.

The proposed N18 Dromoland to Crusheen motorway will cross Latoon Creek upstream of the existing N18 bridge crossing and immediately upstream of Crow's Bridge. The proposed crossing will be single span structure, spanning 55m between abutments. The bridge deck will be arched having a minimum soffit level of 4.9m O.D. (Malin) at the abutments. Realignment of the existing flood levees will be required so as to tie in with the bridge abutments. The main river channel will not be encroached by the structure.

The existing channel upstream of the proposed crossing point has typically a cross-sectional width from levee to levee of approximately 90m and a total cross-sectional flow area at the observed high water mark of approximately 260m^2. The observed high water mark of 3.75m O.D. was measured from trash marks arising from the Christmas '99 flooding which was a combination of extreme river flood flow and high spring tides. The realignment of the levees contracting to a cross section width of 55m will reduce the flow area at the high water mark to approximately 198m^2. The energy losses resulting from this contraction can be determined by multiplying the change in velocity head between the upstream unchanged section and the contracted bridge section, by a contraction coefficient.

$$\Delta y = K_c\left(\frac{u_2^2}{2g} - \frac{u_1^2}{2g}\right) \qquad (1)$$

K_c is a contraction coefficient (0.3 to 0.5 is recommended in the literature), u_2 is the average cross-sectional velocity at the contracted section (bridge section) and u_1 is the average cross-sectional velocity at the upstream unchanged section and Δy is the increase in upstream water level due to the contraction.

The Ardsollus River at Latoon Bridge has a catchment area of 160 km^2 and is tidal for approximately 2km upstream of Latoon Bridge. No gauged flow data exists for the Ardsollus River and extreme flood flows are estimated using the flood Studies methods for ungauged catchments. The annual maximum flood flow can be estimated from the following empirical equation

$$\bar{Q} = 0.00066(\text{Area})^{.92}(\text{SAAR})^{1.22}(\text{SOIL})^2 \qquad (2)$$

where, AREA is catchment area = 160km^2, SAAR is the annual average rainfall amount = 1100mm and SOIL is the soil infiltration potential classification for the site =0.3 (moderate infiltration). The annual maximum flood flow calculated from (2) above is 32.5 m^3/s. The 100 year return period flood flow is estimated by multiplying the annual maximum value of 32.5 by 1.95 (100 year multiplier for Irish rivers) giving 64 m^3/s.

The tidal flow contribution is estimated by determining the difference in reach volumes at high and low waters for the 2km upstream tidal reach and dividing by the ebbing period which is between three and four hours at Latoon Bridge. This gives an average ebb flow rate of 30 to 40 m^3/s. Combining the tide flow with the river flood flow, a total flow of 100 m^3/s is obtained, this represents an extreme flow event at Latoon Creek.

The energy loss (upstream increase in water level) from equation (1) is 0.0027m (2.7mm) using a contraction coefficient of 0.5 for a flow of 100 m^3/s at high tide level. No discernible increase in upstream water level is expected at low tide as the majority of flow will be confined to the main channel and thus the reduction in the overbank flow area due to realignment of levees and bridge abutments will not come into play.

Under existing conditions, flow must contract to pass through Crow's Bridge which has a total width including abutments of 19m. Therefore, the proposed motorway crossing will have much lesser effect on upstream water levels than indicated in the previous paragraph, as flow contraction already exists due to the presence of Crow's Bridge.

Based on trash mark observations the maximum flood high tide water level at the proposed crossing point will not exceed 4.0m O.D. and therefore the proposed bridge soffit level of 4.9mO.D. at the abutments gives sufficient clearance.