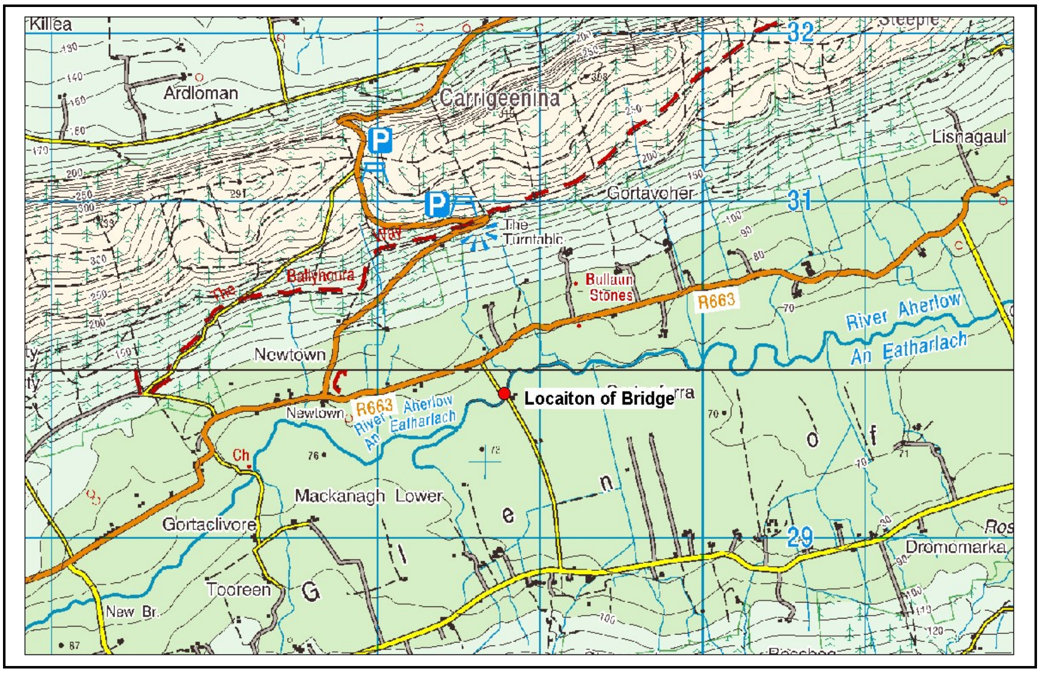
APPLICATION FORM AF-50:
CONSENT UNDER SECTION 50, ARTERIAL DRAINAGE ACT, 1945

APPLICATION DETAILS				
Name of Applicant:				
Company / authority:				
Address:				
Date of Application:				
Client (if appropriate):				
BRIDGE DETAILS				
Bridge Name:	**Stonepark Bridge, Aherlow**			
Purpose (ring appropriate box)	**Public Road**	Private Road	Footbridge	Other
Road Number (or Name):				
River: **Aherlow**		Catchment: Aherlow		
County: **South Tipperary**		Grid Reference: **E:188785 N:129840**		
Location:	**Stonepark, South Tipperary**			
Type of Works (ring appropriate box)	New Bridge	Replacement Bridge	Alterations	



Location Plan

Malachy Walsh and Partners

Park House,21 Denny Street,

CONSULTING ENGINEERS
Cork, Tralee and London

Tralee,Co Kerry.
Telephone 066/7123404
Fax 066/7126586
E-mail: tralee@mwp.ie

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.

Bridge over Aherlow River at Stonepark
Hydrological report

Introduction

South Tipperary County Council proposes to construct a new bridge over the Aherlow River to replace the existing bridge at Stonepark. The existing bridge has an overall span of 19.000m including a central pier consisting of two rectangular columns, each 1.2m wide. This is to be replaced with a single span bridge 18.820m in width. The following is a summary of the design parameters and calculations used to assess the effect of the new bridge on the river and establish a design soffit level.

Location

The new bridge is to be located in the townland of Stonepark in South Tipperary on the Aherlow River as shown on drawing 3485-0020. The river at this location runs through farmland with no residential buildings on the south side of the river in the area. There is a house and some farmyard buildings on the south side of the river immediately downstream of the existing bridge. The land on both sides of the river is subject to frequent flooding and the natural flood plain is clearly visible on both sides at varying distances, which are generally in the order of 100m. The river channel has limited capacity and cannot convey abnormal flows without spilling on the flood plain.

Design

The design flood flows were calculated initially using the Catchment Characteristics Method which is the most appropriate methodology for ungauged catchments greater than 20km^2. The catchment, which has an area of 99 km^2, is outlined on drawing 3485-0021. The Clydagh River, which is contained within the catchment, is a tributary of the Aherlow River and is very steep in relation to the overall catchment slope. The Clydagh valley was, therefore, treated as a sub-catchment and analysed separately from the main catchment. Table 1 shows the data on which the flood flows are based. The mean annual flood for the total catchment to Stonepark based on Catchment Characteristics is 36.41m^3 per second (without application of design factor for standard error).

The Annual Maxima series of flood volumes are available for OPW Station 16007 at Killardry, which is 17km (river length) downstream of the proposed bridge. These are tabulated in Table 2. The total catchment to Killardry is shown on drawing 3485-0022.

A statistical analysis of this data was carried out using an EVI distribution (Table 3) and a Gringorten distribution (Table 4). The resulting flood flows were compared to those calculated using the catchment characteristics data for the total catchment to Killardry (Table 5). The results of the annual maxima series using EVI and Gringorten distributions were higher by factors of 21% and 29% respectively, than the results obtained using the catchment characteristics method.

The results are summarised as follows:

Killardry – mean annual flood Q		**Ratio**
Annual Maxima series – EVI	80.00m^3/sec	1.24
Annual Maxima series – Gringorten distribution	84.70m^3/sec	1.29
Catchment Characteristics method	65.90m^3/sec	1.00

The flow volume to Killardry from the Annual Maxima Series is more conservative, giving a mean annual flood 29% greater, using a Gringorten distribution, than that derived from catchment characteristics. This factor can be used to calibrate the mean annual flood at Stonepark as derived from catchment characteristics. However, since half the data in the Annual Maxima Series at Killardry is greater than the *Limit of Reliability Rating* of 66m^3/sec, the calibration factor has been increased to 1.60 in order to provide an extra allowance for error. On this basis, the flow volumes at Stonepark from catchments characteristics (36.41 ^{3}/sec) have been increased by factor of 1.60 to give a calibrated design mean annual flood of 58.26^3/sec. Table 6 shows the design flood flows for the periods up to 100 years based on this analysis.

River Channel

Drawing 2465-0023 shows the longitudinal profile of the riverbed and the predicted water level profile for the 25-year flood event. Typical cross-sections upstream and downstream of the bridge are shown on drawing 3485-0024. The river channel has an uneven longitudinal profile but has an overall gradient of approximately 1/350. The riverbed consists of sand and gravel and there is some vegetation along the banks.

The flood plain is extensive but quite shallow at Q_{25} levels except for an area close to the northern bank, some distance upstream and downstream of the bridge, which floods to an average depth of 1.35m over a 10m width. This area has been included as part of the cross-sectional area for Q_{25} flows at Section 2 and Section 4. Elsewhere the flood plain has been disregarded in the calculation of the surface profile. Manning Roughness Coefficients of 0.04 and 0.05 have been used for the river channel and flood plain respectively.

Head Loss Through Existing Bridge

For the purpose of analysis the existing bridge has been treated as two separate openings because of the lack of any streamlining in the central piers. The presence of the temporary support is ignored. The parapet walls, soffit and central pier are of rough concrete and the invert is sand and gravel. Assuming a roughness coefficient of 0.005 for the concrete and 0.025 for the riverbed, an overall roughness coefficient of 0.018 (Colebrook-White) has been interpolated for free surface conditions.

At a Q25 flow rate of 93.22m^3/sec, the approach velocity along the upstream channel is 2.70m/sec and the velocity through the bridge is 2.50m/sec. Assuming a 25% loss in the velocity head at entry and exit and a small friction loss, the calculation results are shown in Table 7.

The water surface level is 99.500m on the upstream face of the bridge, which corresponds to the existing soffit level. The presence of flood debris such as trees could cause the level upstream to overtop the bridge and local experience indicates that has occurred on at least one occasion in the past.

Bridge Design

The replacement bridge has been designed with a soffit level 0.300mm higher than that of the existing. It forms a clear span of 18.820 meters between abutments. The new bridge deck has a depth of 0.900m. This is 0.500mm greater than the original and results in an overall increase in road surface level of 0.700mm at the bridge. This increase is the maximum that can be allowed while still maintaining a reasonable vertical profile on the public road. The absence of the central pier will reduce the possibility of accumulation of debris, which may have caused increased upstream flood levels in the past. The existing bridge parapet railings will be replaced with solid masonry walls. This is an important aesthetic feature of the replacement bridge given its location and setting in the Glen of Aherlow. The hydraulic implications of the new bridge design are discussed below.

Head loss through new bridge

The new bridge will have a single span of 18.820 meters. The parapet walls and soffit will be of smooth concrete and the river will be of sand and gravel. Assuming a roughness coefficient of 0.0015 for the smooth concrete and 0.025 for the riverbed, an overall roughness coefficient of 0.020 (Colebrook-White) has been interpolated for free surface conditions.

The Q_{25} velocity through the new bridge is slightly lower than that through the existing bridge at 2.36m/s. The approach velocity is 2.70m/s. Assuming a 25% loss in velocity head at entry and exit and a small friction loss through the bridge, the calculated difference in surface level across the new bridge is 0.170m. The calculation results are shown in Table 8. The resulting water surface level on the upstream face of the new bridge is 99.480m, leaving a 0.320m freeboard to the soffit.

The head difference due to the new bridge is quite low and is slightly less than that of the existing bridge. The water level in the vicinity of the bridge is controlled primarily by the downstream channel, with the bridge causing a minimal backwater effect upstream.

Flood flows of exceptional magnitude may exceed the new soffit level and result in surcharging of the bridge. As stated earlier, the new bridge is designed with stone parapet walls in place of the railings used in the existing and this will prevent any possible flow over the bridge deck. The bridge has been designed to resist lateral loading due to surcharge. South Tipperary County Council have been made aware of the possibility of flood levels in excess of the new soffit level for extreme flood events.

Summary

The new bridge has been designed to cater for flood events of 25-year return period. The design soffit level is 99.800m, which is 0.300m higher than that of the existing bridge. The freeboard for the Q_{25} flood event is 0.320m.

List of Tables

List of Drawings

Data	Units	Value	
Catchment		Aherlow	Clydagh
Catchment Area	km^2	88.25	10.75
Stream Length	km	25.90	6.16
Level@10% distance	m	75.00	86.00
Level@85% distance	m	175.00	353.00
Stream Slope S1085	m/km	5.15	57.79
C coefficient		0.0172	0.0172
Stream Frequency STMFRQ	junctions/km^2	0.87	2.25
Soil Class 1	km^2	0.00	0.00
Soil Class 2	km^2	32.00	0.00
Soil Class 3	km^2	0.00	0.00
Soil Class 4	km^2	15.25	0.60
Soil Class 5	km^2	41.00	10.15
Soil Index		0.42	0.50
SAAR	mm	1360	1360
M5-2day	mm	75	90
M5-60min / M5-2day	mm	0.225	0.20
M5-24hour / M5-2day	mm	0.805	0.79
M5-24hour	mm	60.38	71.10
M5-1day	mm	54.39	64.05
ARF		0.945	0.975
SMDBAR	mm	5.00	5.00
RSMD	mm	46.40	57.45
Calculated Q_{mean}	m^3/sec	25.92	10.49
Total calculated Q_{mean}	**m^3/sec**	**36.41**	

Table 1 – Catchment Analysis at Stonepark (without application of design factor for standard error)

Record No	Year	Annual max Q
1	1954	102.00
2	1955	57.80
3	1956	138.00
4	1957	99.80
5	1958	64.80
6	1959	65.80
7	1960	88.10
8	1961	63.80
9	1962	72.60
10	1963	95.10
11	1964	76.90
12	1965	110.00
13	1966	63.30
14	1967	46.50
15	1968	136.00
16	1969	41.60
17	1970	59.70
18	1971	91.60
19	1972	51.10
20	1973	86.40
21	1974	44.90
22	1975	91.60
23	1976	38.60
24	1977	68.90
25	1978	93.90
26	1979	83.00
27	1980	61.70
28	1981	87.00
29	1982	75.80
30	1983	60.00
31	1984	45.60
32	1985	93.30
33	1986	53.00
34	1987	72.40
35	1988	103.00
36	1989	127.00
37	1990	41.60
38	1991	90.40
39	1992	52.20
40	1993	68.40
41	1994	107.00
42	1995	84.70
43	1996	103.00
44	1997	113.00
45	1998	109.00
46	1999	56.80
47	2000	113.00

Mean	79.23
Standard Deviation	25.26
u	67.86
α	19.70

Return Period	Reduced Variate	Q (m^3/sec)
2.4	0.618	80.0
5	1.500	97.4
10	2.250	112.02
25	3.199	130.9
50	3.902	144.7
100	4.600	158.5

Table 2 – Annual maxima Series at Killardry

Rank No	Year	Annual Max Q (m^3/s)	Return Period
1	1956	138.00	84.14
2	1968	136.00	30.21
3	1989	127.00	18.41
4	1997	113.00	13.24
5	1965	110.00	10.33
6	1998	109.00	8.47
7	1994	107.00	7.18
8	1996	103.00	6.23
9	1988	103.00	5.50
10	1954	102.00	4.93
11	1957	99.80	4.46
12	1963	95.10	4.08
13	1978	93.90	3.75
14	1985	93.30	3.47
15	1975	91.60	3.24
16	1971	91.60	3.03
17	1991	90.40	2.85
18	1960	88.10	2.68
19	1973	86.40	2.54
20	1995	84.70	2.41
21	1979	83.00	2.29
22	2000	81.52	2.19
23	1999	81.42	2.09
24	1981	78.00	2.00
25	1964	76.90	1.92
26	1982	75.80	1.84
27	1962	72.60	1.77
28	1987	72.40	1.71
29	1977	68.90	1.65
30	1993	68.40	1.59
31	1959	65.80	1.54
32	1958	64.80	1.49
33	1961	63.80	1.45
34	1966	63.30	1.40
35	1980	61.70	1.36
36	1983	60.00	1.33
37	1955	57.80	1.29
38	1986	53.00	1.25
39	1992	52.20	1.22
40	1972	51.10	1.19
41	1970	49.70	1.16
42	1967	46.50	1.13
43	1984	45.60	1.11
44	1974	44.90	1.08
45	1990	41.60	1.06
46	1969	41.60	1.03
47	1976	38.60	1.01

Table 4 – Maxima Series at Killardry – Gringorten Distribution

Data Description	Units	Value
Catchment		Killardry
Catchment Area	km^2	283.30
Stream Length	km	43.32
Level@10% distance	m	75.00
Leavel@85% distance	m	145.00
Stream Slope	m/km	2.15
C coefficient		0.0172
Stream Frequency STMFRQ	junctions/km^2	0.92
Soil Class 1	km^2	0.00
Soil Class 2	km^2	103.60
Soil Class 3	km^2	0.00
Soil Class 4	km^2	61.50
Soil Class 5	km^2	118.2
Soil Index		0.42
SAAR	mm	1360
M5-2day	mm	75
M5-60min / M5-2day (r)	mm	0.220
M5-24hour / M5-2day	mm	0.805
M5-24hour	mm	60.38
M5-1day	mm	54.39
ARF		0.920
SMDBAR	mm	5.00
RSMD	mm	45.04
Calculated Q mean	m^3/sec	65.90

Table 5 – Catchment Analysis at Killardry (without application of design factor for standard error)

Return Period (years)	Q_T/Q_{mean}	Calibration Factor	Flow to Stonepark (m^3/sec)
2.4	1.00	1.60	58.26
5	1.20	1.60	69.92
10	1.37	1.60	79.82
25	1.60	1.60	93.22
50	1.77	1.60	103.13
100	1.96	1.60	114.20

Table 6 – Design flows at Stonepark

Section	Flow Area (m^2)	Perimeter (m)	Hydraulic Radius (m)	Slope	Roughness Coefficient	Velocity (m/sec)	Discharge per opening (m^3/sec)	No of opes	Total Discharged (m^3/sec)	Bridge length (m)	Head Loss (m)	Total Loss (m)
Approach						2.70					0.093	
Existing Bridge	18.75	13.14	1.43	0.00147	0.018	2.50	46.61	2	93.22	5.000	0.007	
Exit						2.50					0.080	0.180

Table 7 – Head Loss through existing bridge

Section	Flow Area (m^2)	Perimeter (m)	Hydraulic Radius (m)	Slope	Roughness Coefficient	Velocity (m/sec)	Discharge per opening (m^3/sec)	No of opes	Total Discharged (m^3/sec)	Bridge Length (m)	Head Loss (m)	Total Loss (m)
Approach						2.70					0.093	
Existing Bridge	39.48	23.00	1.72	0.00107	0.020	2.36	93.22	1	93.22	6.100	0.007	
Exit						2.36					0.070	0.170

Table 8 – Head Loss through new bridge